

CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

SEPTEMBER 1958



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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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Volume LIII, No. 9.

LONDON, SEPTEMBER, 1958.

EDITORIAL NOTES

Above the Law.

STATEMENTS made in the House by Members of Parliament are not actionable in law, however badly a citizen may think that he has been damaged or libelled by a statement made by a privileged person in a privileged place. The same immunity is enjoyed by newspapers in reporting the proceedings of Parliament, with the result that defamatory remarks by Members may receive unrestricted publicity. The Speaker of the House of Commons makes strenuous efforts to prevent members from reflecting on the honour of other members, but the rules of the House do not seem to empower him to stop Members from making charges that may prove to be untrue and that may cause damage and suffering to people outside the House. For this reason most Members are careful thoroughly to investigate matters that they think should be brought to the attention of the House, and through the House to the nation, before they make such allegations. Such care does not seem to be exercised by the Members who constitute the Committee of Public Accounts appointed by Parliament to investigate instances of what they consider to be waste, extravagance, or other misuse of public money by Government departments. Further, the House orders the reports of the Committee to be published apparently without taking the elementary duty of a publisher of making sure that what he publishes is accurate and truthful.

In a report issued in July 1957 the Committee dealt with the cost of the radio telescope built at Jodrell Bank, Cheshire, for Manchester University, and for which the Government provided a large part of the cost. The Committee found that the cost of the telescope had risen from an estimate of £439,616 made in 1953 to £700,000 in 1957. In 1955 the University informed the Department of Scientific and Industrial Research of an extra cost of about £240,000, and a committee of inquiry was set up. In a summary of the report of this committee submitted to the Treasury by the Department of Scientific and Industrial Research it was stated that "a very unsatisfactory position between the University and their consultants" had been revealed. The Committee of Public Accounts then held its own inquiry and was told that the consulting engineers changed the design without the concurrence of the University, that new structural features were introduced into the instrument and the complexity and the cost thereby increased, and that the University professor who was primarily responsible for

the design was not consulted so far as the D.S.I.R. was aware. So far the matter was discussed privately, but in its published report the Committee of Public Accounts stated that "they were particularly surprised to be told that the consulting engineers before introducing substantial modifications in a novel and costly scientific instrument had not followed the common-sense course of discussing their plans with the eminent scientist who was in charge of the project", and added the comment that they "regarded as highly unsatisfactory a state of affairs in which it was possible for the project to be altered at greatly increased cost without the consent and approval of the Department of Scientific and Industrial Research or of the University, or even of discussion with the scientist in charge".

This statement received widespread publicity coupled with the name of the consulting engineers concerned. It was read with amazement by those who knew the record of this well-known firm of engineers and of the many important works for which they have been responsible at home and abroad. It is likely that prospective clients noted that it might not be safe to entrust work to engineers who were alleged in a Parliamentary document to have added £240,000 to the cost of a structure without having enough common sense to ask permission or even to advise their client of their intention to do so. There are few allegations that could do more shattering injury to a consultant. Yet a year later, in a report published in July 1958, the Committee admits that its published statements of a year before were untrue. The latest report states that "it is now clear that there was in fact the fullest collaboration" between the consulting engineers and the professor who was acting on behalf of the University. The meaning of "fullest collaboration" is not defined, but as the Committee now admits that its previous report was "gravely inaccurate and misleading" it is fair to assume that the improvements suggested by the consultants were accepted by the professor on behalf of the University before they were implemented. No apology is offered to the consultants who have for a whole year suffered from the inaccurate statements contained in the previous report. So can an engineer be libelled without redress.

It is very disturbing to know that such a damaging report can be published without both sides of a question being heard, for it seems that the report was based on the evidence only of the Department of Scientific and Industrial Research and the Treasury. It is not recorded that the professor or the consultants were asked to give evidence, although either of them could have prevented this grave injustice being done. In most of the reports of this committee the people who are censured are established Civil Servants whose names do not become known and whose careers are not affected. It is a vastly different matter when the victim of inaccurate and misleading statements is a private citizen whose name is known and whose livelihood and the respect of his fellows can be taken from him as a result of false insinuations. It is likely that this case will result in Civil Servants taking care that the evidence they give to the Committee is strictly true, but it seems even more important that the Committee should hear all sides of a question before publishing statements that the youngest journalist would know should not be published without the fullest confirmation of their truth in every detail, and which, if they were published without the immunity conferred upon these reports by Parliament, would result in a large sum of money being awarded by the Courts as compensation to the aggrieved party.

A Large "Shell" Dome.

A DOUBLY-CURVED dome, 125 ft. square, which has recently been built to cover two tennis courts at the All-England Tennis Club, Wimbledon, is shown in *Figs. 1 to 3*.

A structure which would provide the greatest amount of natural lighting, with sides consisting of folding or sliding doors, was required, and it was preferred that the roof should have no internal supports.

The area to be covered by the roof was 15,200 sq. ft. Three types of structure were considered, namely, steel roof trusses and framework, twin cylindrical concrete shells, and a doubly-curved concrete shell. The last of these was chosen because of its low cost and architectural and functional merits. It was originally proposed to extend the arches to ground

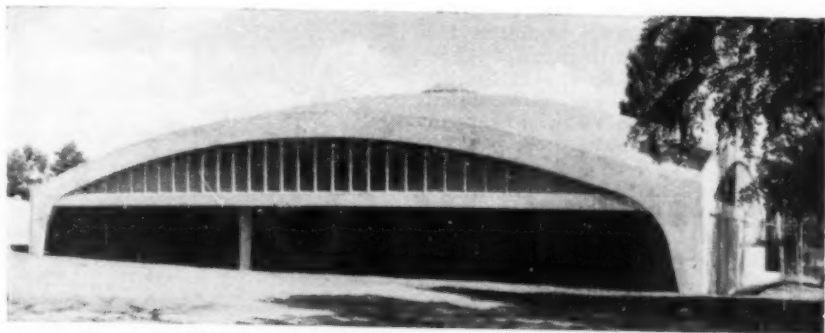


Fig. 1.—View of Side with Folding Doors.

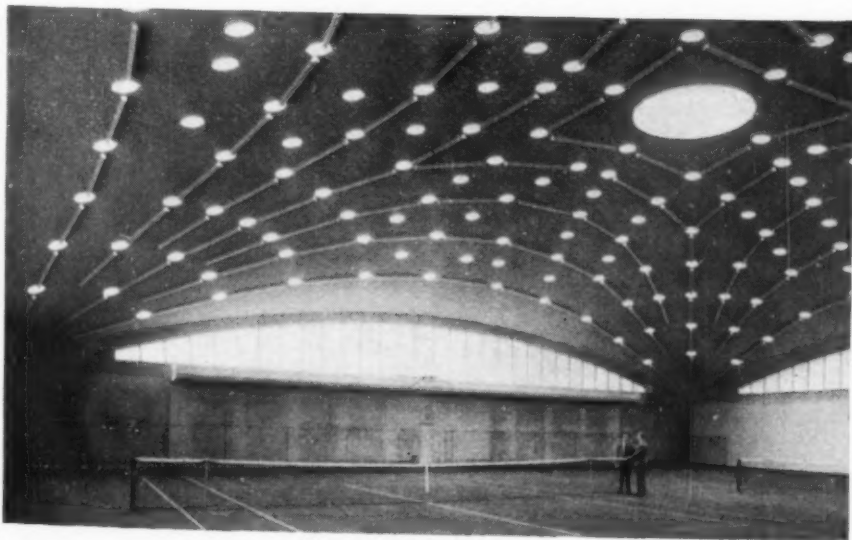


Fig. 2.—Interior View.

level, and resist the horizontal thrusts by means of raking piles. This solution, however, would have caused too much obstruction, and the boundary members of the shell are supported on four corner columns. It was also decided to build two of the walls in brickwork, thereby allowing two balconies with seating accommodation for about 100 people to be provided. The balconies are cantilevered from columns in the brick walls, and are structurally independent of the dome.

The shell is 3 in. thick except for a width of 11 ft. 6 in. along each side in which the thickness is increased (Fig. 4), and has the form of a paraboloid with a span of 175 ft. and a rise of 24 ft. 4½ in. Four segments with chords of 124 ft. 3 in. were omitted from the circular projection, providing a square plan and segmental openings along all sides.

Natural lighting is provided by the glazed segmental openings, 200 small roof lights with internal diameters of 1 ft. 9 in. placed along the trajectories of minimum

principal stress, and a central lantern 14 ft. in diameter. Precast concrete mullions are used in the segmental openings. Artificial lighting with an intensity of 40 lumens per square foot is provided by means of fluorescent lamps placed along the trajectories of maximum principal stress, and the wiring necessary to increase the intensity to 60 lumens is incorporated.

Design.

The design of the dome is based on the assumption that the shell is a stressed membrane, and that the supporting arches resist only the forces acting in their own planes. For convenience, the analysis was transformed into the horizontal plane so that the stress resultants could be expressed in terms of the stress function $F(x,y)$ as follows.

Direct forces:

$$n_{xx} = \frac{\partial^2 F}{\partial y^2}; \quad n_{yy} = \frac{\partial^2 F}{\partial x^2}.$$

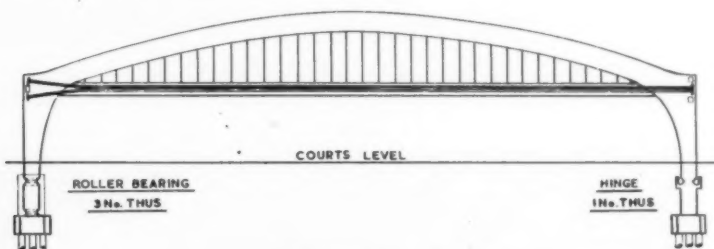


Fig. 3.—Cross Section.

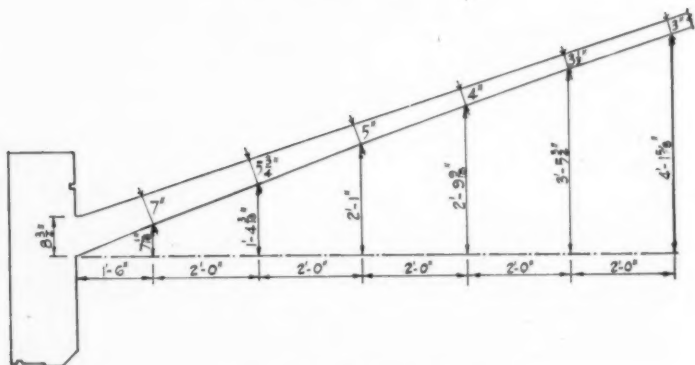


Fig. 4.—Part Cross Section.

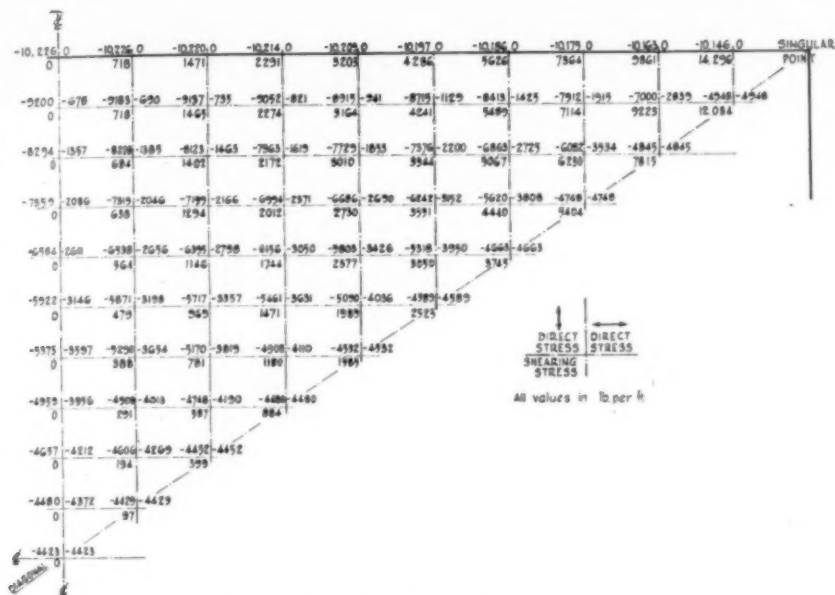


Fig. 5.—Stress Resultants for Part of Dome.

Shearing forces:

$$n_{xy} = n_{yx} = - \frac{\partial^2 F}{\partial x \partial y^2}$$

The stress function $F(x,y)$ must satisfy the differential equation

$$\frac{\partial^2 f}{\partial x^2} \cdot \frac{\partial^2 F}{\partial y^2} - 2 \frac{\partial^2 f}{\partial x \partial y} \cdot \frac{\partial^2 F}{\partial x \partial y} + \frac{\partial^2 f}{\partial y^2} \cdot \frac{\partial^2 F}{\partial x^2} = Z$$

in which $f = f(x,y)$ is the equation of the middle surface of the shell and $Z = Z(x,y)$ is the specific value per unit area at the point $f(x,y)$ of the system of uniformly-distributed forces acting on the shell. Solutions of the equation, and hence the stress-resultants, were obtained at the points shown in Fig. 5. The normal and diagonal symmetry of the structure made it necessary to consider only one-eighth of the surface area of the shell; the results obtained are shown in Fig. 5. Loads due to wind are allowed for only in the design of the columns, as the stresses due to wind in a doubly-curved shell are known to be negligible. The reinforcement is shown in Figs. 6 and 7.

Construction.

The formwork for a doubly-curved shell of this type is simple, since a paraboloid can be generated either by the rotation of a parabola around its vertical axis or by the translation of the parabola along an identical curve, both curves remaining vertical. The supports for the formwork could therefore have been made to consist of parabolas intersecting radially at the crown, concentric parabolas at varying levels, or identical parabolas at varying levels in parallel and equidistant vertical planes. The last of these methods was used because of its simplicity and economy.

The curvature of the shell is small. It was at one time intended that the scaffold tubes forming the generating tubes (Fig. 8) should be bent to three different radii; a single radius of 166 ft. 3 in. was, however, found to be suitable, the variations of curvature being obtained by springing the tubes during fixing. The tennis courts, which were already in existence, were covered with a layer of building paper and with boards on which timber bearers were placed at centres of

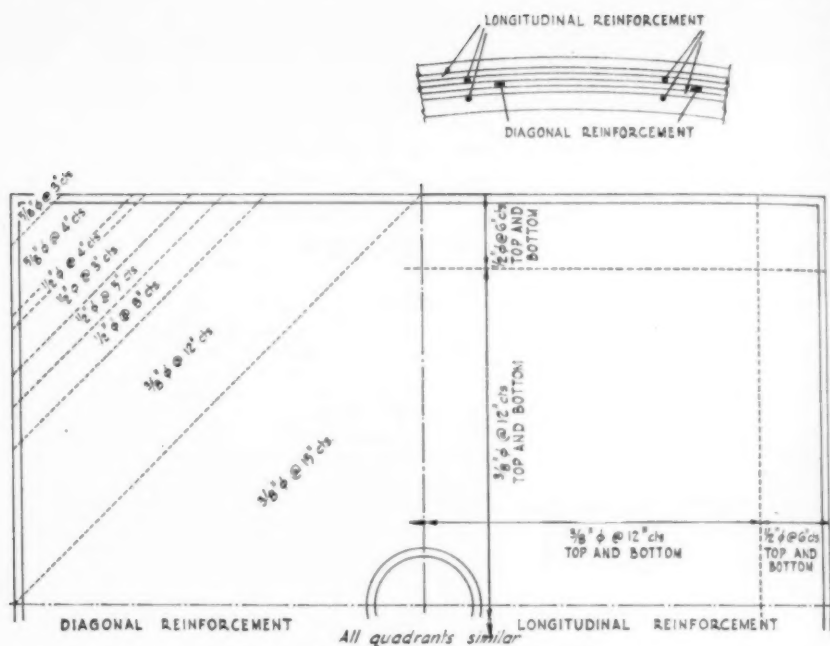


Fig. 6.—Arrangement of Reinforcement.

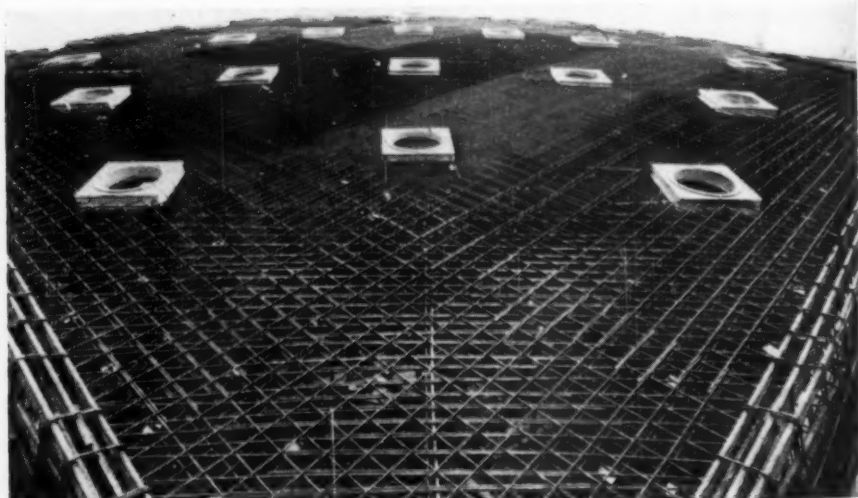


Fig. 7.—Reinforcement for Dome.

6 ft. On these bearers vertical scaffold tubes were placed at centres of 6 ft. in both directions and a datum line was marked on each tube. The heights of the tubes forming the generating curves were measured from these lines and checked by means of parabolic steel jigs to ensure that the surface obtained was correct to within $\frac{1}{8}$ in. Curved tubes at centres of 1 ft. were then placed over the generating curves (Fig. 9) to support the expanded-metal formwork. The scaffold included 22,000 ft. of curved tubes and 53,000 ft.

was sprayed on the underside of the roof for acoustical purposes. The remainder of the shuttering consisted of plywood $\frac{3}{4}$ in. thick.

The aggregate consisted of Thames gravel, with a maximum size of $\frac{3}{4}$ in., and washed sand. Details of the concrete are given in Table I; the water-cement ratio was not specified, a slump of 1 in. being used throughout.

An 18/12 revolving-drum mixer with a built-in weigher and hand-operated loading shovel was used. The aggregates

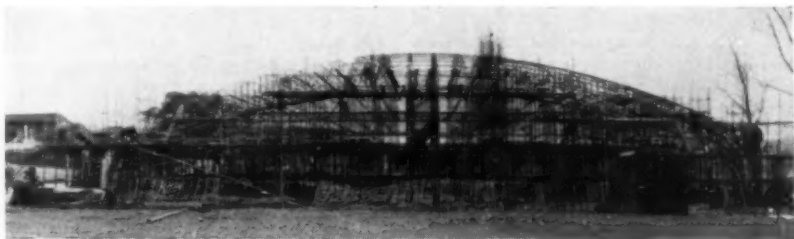


Fig. 8.—Erection of Steel Scaffolding.

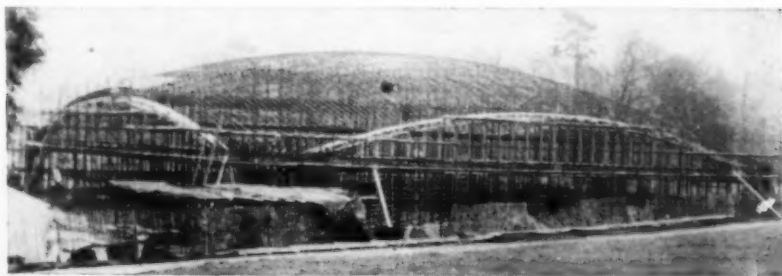


Fig. 9.—Scaffolding Complete.

of straight tubes. After the completion of the roof the curved scaffolding was removed, the remainder forming a platform from which the underside of the roof was sprayed.

Expanded-metal formwork (Fig. 10) instead of timber was used for the roof for the following reasons. (1) It was necessary to make the formwork for the entire roof at once. (2) A paraboloidal surface cannot be developed, and therefore, particularly at the corners, special fitting was unavoidable. (3) The expanded metal formed a key for a finish $\frac{1}{8}$ in. thick which

and cement were delivered by road. The cement was delivered loose and stored in a silo. The mixed concrete was tipped into a dumper with a capacity of $\frac{1}{2}$ cu. yd. in which it was transported to one of four hoists placed at the middle of each side of the dome, whence it was barrowed to the required position.

An important problem in the design and construction of the roof was the reduction as much as practicable of the effects of plastic yield, shrinkage, and differential settlement. It was impossible to predict these exactly, and they may

TABLE I.—PROPORTIONS AND STRENGTHS OF CONCRETE.

Proportions	Specified minimum cube strength (lb. per square inch)		Average cube strength obtained (lb. per square inch)		Where used
	At 7 days	At 28 days	At 7 days	At 28 days	
1 : 2 : 4	2000	3000	3700	5000	Foundations, roof, balconies.
1 : 1½ : 3	2500	3750	5000	6500	Arches, tie beams, edge of roof.
1 : 1 : 2	3000	4500	7000	8000	Columns.

have seriously affected the distribution of stress in a structure of this magnitude. The boundary members were therefore rendered statically determinate until after the dead load had been applied. The members are tied arches, the ties comprising prestressing cables placed in ducts in concrete beams.

The method of construction was as follows. The precast mullions were placed in position, and the arch was cast. The mullions, which at this stage were suspended from the arch, were used to support a trough made of expanded metal, in which the prestressing cables were placed. The four inner cables were placed in metal sheaths; the upper and lower cables were left uncovered. The upper and lower

cables were tensioned, and the prestressing force so applied was sufficient to lift the roof clear of the scaffolding, thereby transferring the load to the columns at the four corners. Three of the columns rested on roller bearings of the type shown in Fig. 11, comprising compressive members hinged at the top and bottom; the fourth column rested on a fixed bearing with one hinge only. The hinges were circular in plan, and by this means bending moments on the columns were avoided.

The trough was then filled with grout, the untensioned cables being protected by the sheathing, and the tie-beam was shuttered and cast. The remainder of the prestress was applied, thereby introducing one redundancy in each arch caused by

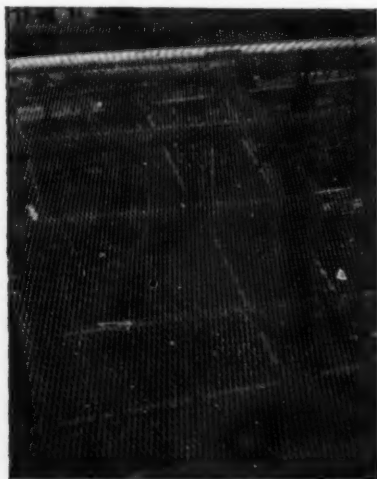


Fig. 10.—Expanded Metal used as Centering.

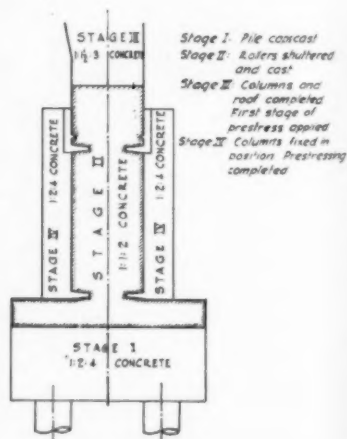


Fig. 11.

the tie-beam and the prestressing force, and the inner cables were grouted. Finally, the columns were fixed in position by placing concrete in and around the hinges (Fig. 11), thereby introducing a second redundancy for live loads and temperature stresses. Eight cables, each comprising twelve 0.276-in. wires, were used in each tie-beam.

The sequence of casting the roof is shown in Fig. 12. The concreting of the roof was started in January 1958 and completed two months later.

Because of the poor quality of the ground each column is supported on six friction piles, 17 in. in diameter and 65 ft. long. The dome is covered with roofing felt.

The cost of the structure and finishes was about £40,000. The consulting engineers were Messrs. C. J. Pell & Partners, and the contractors were Messrs. W. H. Gaze & Sons, Ltd., in conjunction

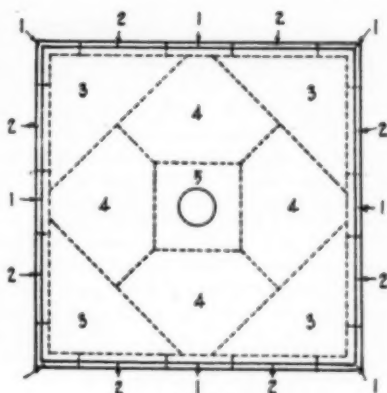


Fig. 12.

with Formcrete, Ltd. The scaffolding was supplied and erected by the Steel Scaffolding Co., Ltd.

Pile Calculations by the Wave-Equation.

MR. DONOVAN H. LEE, M.I.C.E., writes as follows.

The most interesting article by Mr. E. A. Smith in your June issue reminds us that little attempt has been made until recently to apply the results of the research on maximum stresses in piles during driving made at the Building Research Station in 1936-8 to determine resistance to friction. However, the time needed to calculate this resistance by the stress-wave theory raises the question whether a simple formula or graphical method might be developed to follow the theory with tolerable accuracy, and be more nearly correct than any impact formula now in use. The writer is tempted to believe, as a result of studies made recently, that this is possible, but there would need to be important qualifications for short piles and assumptions would have to be made regarding the part of the resistance to penetration provided by side friction.

As Mr. Smith has pointed out, the resistance to penetration at the time of driving is not necessarily equal to the ultimate static load the pile can sustain, and it is important to make proper allowance for the expected causes of difference before applying the appropriate factor of

safety. I refer, of course, to the need to consider piles also as a group and the possible consolidation with time in the case of cohesive soils, as well as "take-up" and the opposite. Also, when piles are being driven against the very high resistance of confined waterlogged soil, as may occur with quicksand and with clay that cannot be displaced vertically or laterally, the resistance to penetration may have no relation to the ultimate static load.

For these reasons it is my opinion that the safe load on driven piles must continue to be assessed in relation to the properties of the soils into which they are driven. The stress-wave theory, or an impact formula, is, however, most useful for determining whether, and how, the soil varies over the site, and in the case of granular soils to indicate that the desired safe load will be achieved. The stress-wave theory also demonstrates the effect of unusual conditions, which could not be expected from an impact formula.

[The "take-up", or increase in friction after driving, although often correctly treated as a permanent increase in ultimate static load, may in the case of soils which will consolidate later sometimes need to be treated as a cause of extra load on the foot of the pile.]

Book Reviews.

"Pre-Stressed Concrete." By R. H. Evans and E. W. Bennett. (London: Chapman & Hall, Ltd. Price 6os.)

THIS book comprises three parts: the first deals with the analysis of simply-supported beams, the second with the design of such beams, and the third with other applications of prestressing.

In the first part the theory of prestressed beams is well presented in concise yet comprehensive manner, and many examples are included. Two designs are given in the second part. The application of theory is clearly described, but no attempt is made, either here or elsewhere in the book, to include practical details or to discuss the merits of different methods of prestressing. The third part is uneven; for example prestressed piles are dealt with in only $1\frac{1}{2}$ pages, but liquid-retaining structures and prestressed domes and shells are considered in much greater detail. The book is clearly written and well produced.

"Proceedings of the Second Congress of the Fédération Internationale de la Précontrainte." 1000 pages. (Obtainable from Concrete Publications, Ltd., London. Price £5; by post £5 2s. 6d. (in North America \$20).)

MORE than forty papers and communications, together with the discussions thereon, which were contributed by specialists in prestressed concrete from thirty countries, are given in full with many illustrations in this report of the second international congress of the Federation held in Amsterdam. The papers are printed in the language in which they were presented, that is English, French, or German, and the general reports and communications are printed in all three languages. The principal subjects dealt with are:

Effect of grouting and anchorages on the behaviour of prestressed members.

The results of experience in the production and use of prestressing steel.

Progress in the production of prestressed precast members in factories.

Prestressing of precast units on construction sites.

Redistribution of moments in prestressed statically-indeterminate structures loaded beyond the elastic stage.

Influence of plasticity on the strength and stability of thin prestressed shells.

Analysis of practice in prestressed concrete in various countries.

Economical advantages of prestressed concrete.

Method of direct reading of the loss of stress due to creep.

The economies of prestressed pressure pipes.

Prestressed arches.

Industrial structures in prestressed concrete.

The use of expansive mortar for prestressing.

Beams subjected to bending and torsion.

The volume was prepared in London by the Cement and Concrete Association, by whom it has been recently issued.

"Safe Loads and Properties for Square Tied Columns." By A. d'o Smith. (Published by the author. Price 3s. 9d.)

THIS booklet of 18 pages gives tables of safe axial loads, equivalent areas, and section moduli for square columns reinforced with mild steel or deformed bars.

Books Received

"Rakennuskustannukset Helsingissa" (Building Costs in Helsinki), by Pentti Pöyhönen. Price 500M.

"Värmisoleringsmaterial och Korrosion", by Tenho Sneek and Eero Hänninen. No price stated.

"Om Förändringar i Myosinfraktioner ur Fiskmuskel vid Nedfrysning, Frysagring och Upptining", by Olavi E. Nikkilä. No price stated.

"Omakotirakennuksen Suunnittelun Taloudelliset Perusteet", by P. O. Jarle. Price 1250M. (Helsinki: The State Institute for Technical Research.)

"A Classification of Danish Flints, etc., based on X-ray Diffractometry", by A. Tovborg Jensen, C. J. Wohlk, K. Drenck, and E. Krogh Andersen. [Copenhagen: Danish National Institute of Building Research. Price 12 Kroner.]

"Sul Comportamento dei Refrattari ad Elevato Tenore in Allumina nei Forni Rotanti da Cemento." In two parts. By Antonio Cocco. (Trieste: Università degli Studi di Trieste, Facoltà di Ingegneria. Price not stated.)

Determination of Fixed-end Moments for Prismatic Members.

By J. C. STEEDMAN.

It is often necessary to obtain the fixed-end moments caused by partial uniform or triangular loads. The charts given in this article enable these moments to be determined rapidly and with an accuracy which is generally sufficient. If greater accuracy is required, the charts may be used for checking. The moments are given in the form of a coefficient times span times total load on the span. Values for most combinations of loads can be obtained by superposition, and the position of a partial uniform or triangular load which causes the greatest possible fixing moment can also be obtained.

Derivation of Formulæ.

The following formulæ are given by Mr. Shepley.*

Partial uniform load :

$$\bar{M}_A = \frac{W}{12L^2} [12ab(b+c) + 6cb^2 + 4c^2(a+b) + c^3] \quad (1)$$

Partial triangular load :

$$\bar{M}_A = \frac{wc}{60L^2} [10bc(b+c) + 15a(2b^2+c^2) + 40abc + 3c^3] \quad (2)$$

$$\bar{M}_B = \frac{wc}{60L^2} [20ac(a+b) + 5c^2(2a+b) + 30a^2b + 2c^3] \quad (3)$$

Applied moment :

$$\bar{M}_A = \frac{b}{L} \left(2 - 3\frac{b}{L} \right) M \quad (4)$$

$$\bar{M}_B = \frac{a}{L} \left(2 - 3\frac{a}{L} \right) M \quad (5)$$

Replacing c by x and b by $(1-a)$, and expressing a and x in terms of L :

$$\bar{M}_A = WL \left(a^3 - 2a^2 + a + \frac{3a^2x}{2} - 2ax + ax^2 + \frac{x}{2} - \frac{2x^2}{3} + \frac{x^3}{4} \right) \quad (1A)$$

$$\bar{M}_A = WL \left(a^3 - 2a^2 + a + a^2x - \frac{4ax}{3} + \frac{ax^2}{2} + \frac{x}{3} - \frac{x^2}{3} + \frac{x^3}{10} \right) \quad (2A)$$

$$\bar{M}_B = WL \left(-a^3 + a^2 - a^2x + \frac{2ax}{3} - \frac{ax^2}{2} + \frac{x^2}{6} - \frac{x^3}{10} \right) \quad (3A)$$

$$\bar{M}_A = M(3a^2 - 4a + 1) \quad (4A)$$

$$\bar{M}_B = M(2a - 3a^2) \quad (5A)$$

and by selecting suitable values of a and x Charts 1 to 4 are obtained.

* "Continuous Beam Structures", by E. Shepley. Concrete Publications, Ltd.

CHART I.

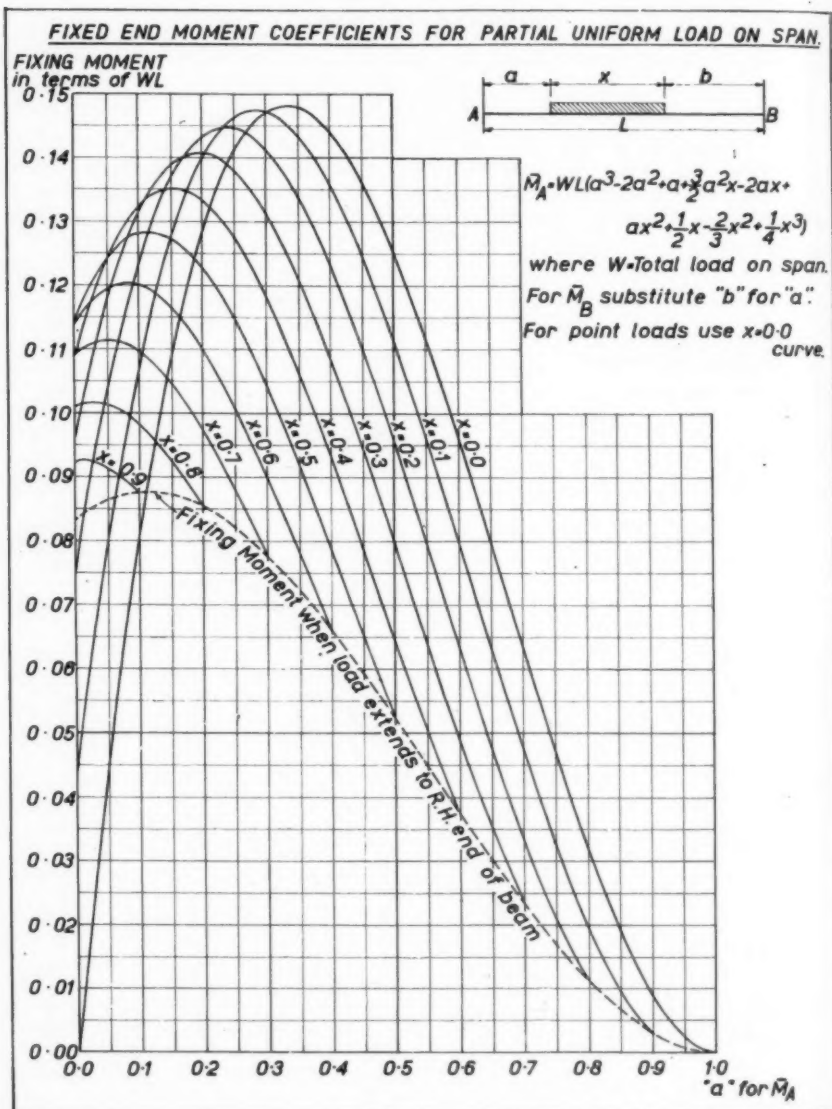


CHART 2.

FIXED END MOMENT COEFFICIENTS FOR PARTIAL TRIANGULAR LOAD ON SPAN.

FIXING MOMENT $\bar{M}_A = WL(a^3 - 2a^2 \cdot a + a^2 x - \frac{4}{3}ax + \frac{1}{2}ax^2 - \frac{1}{3}x - \frac{1}{3}x^2 + \frac{1}{10}x^3)$. SHEET 1. \bar{M}_A in terms of WL .

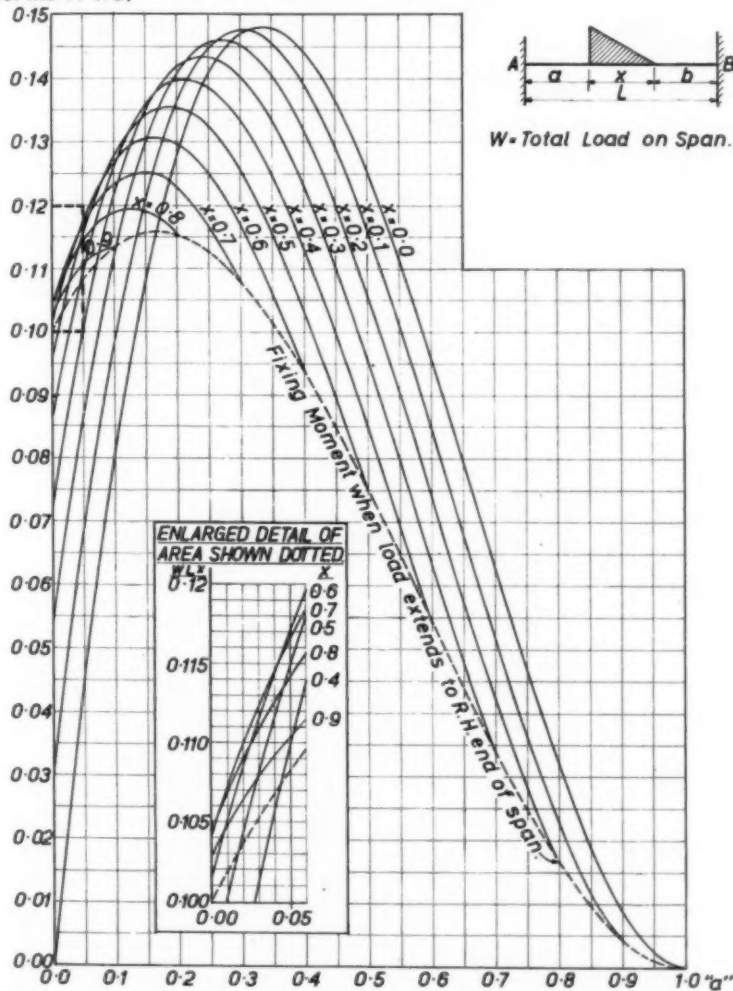


CHART 3.

FIXED END MOMENT COEFFICIENTS FOR PARTIAL TRIANGULAR LOAD ON SPAN.
 $M_B = WL(-a^3 + a^2 - a^2x + \frac{2}{3}ax - \frac{1}{2}ax^2 + \frac{1}{6}x^2 - \frac{1}{10}x^3)$ **SHEET 2.M_B**

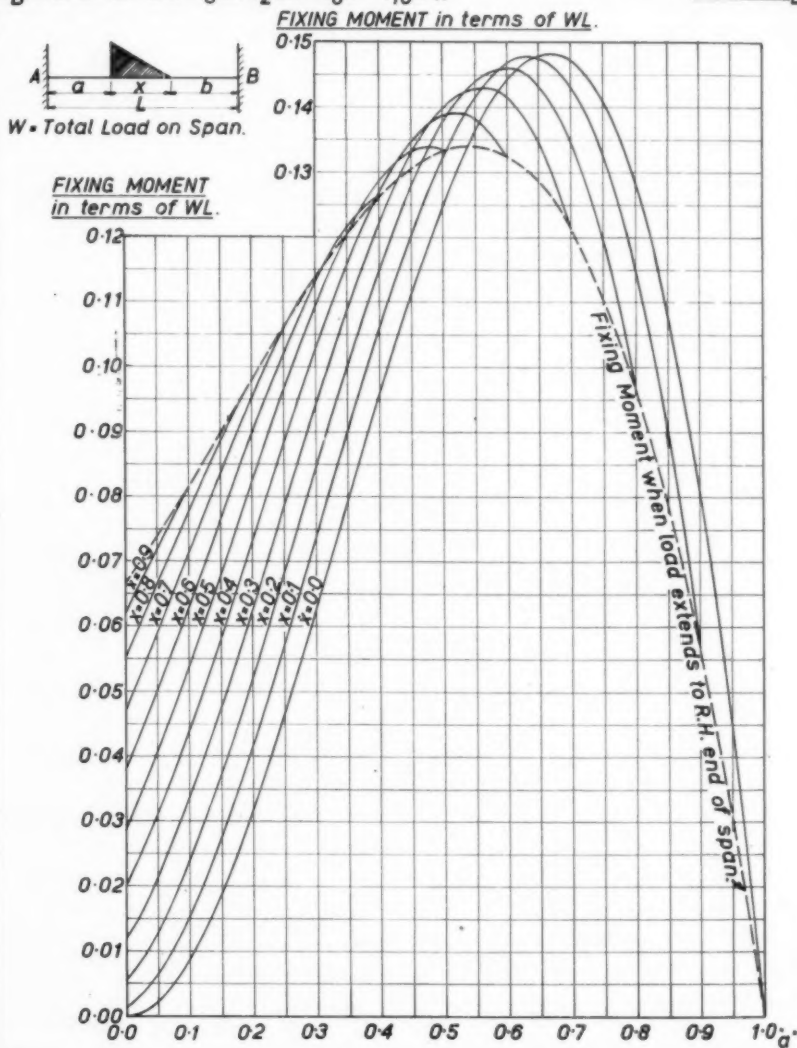
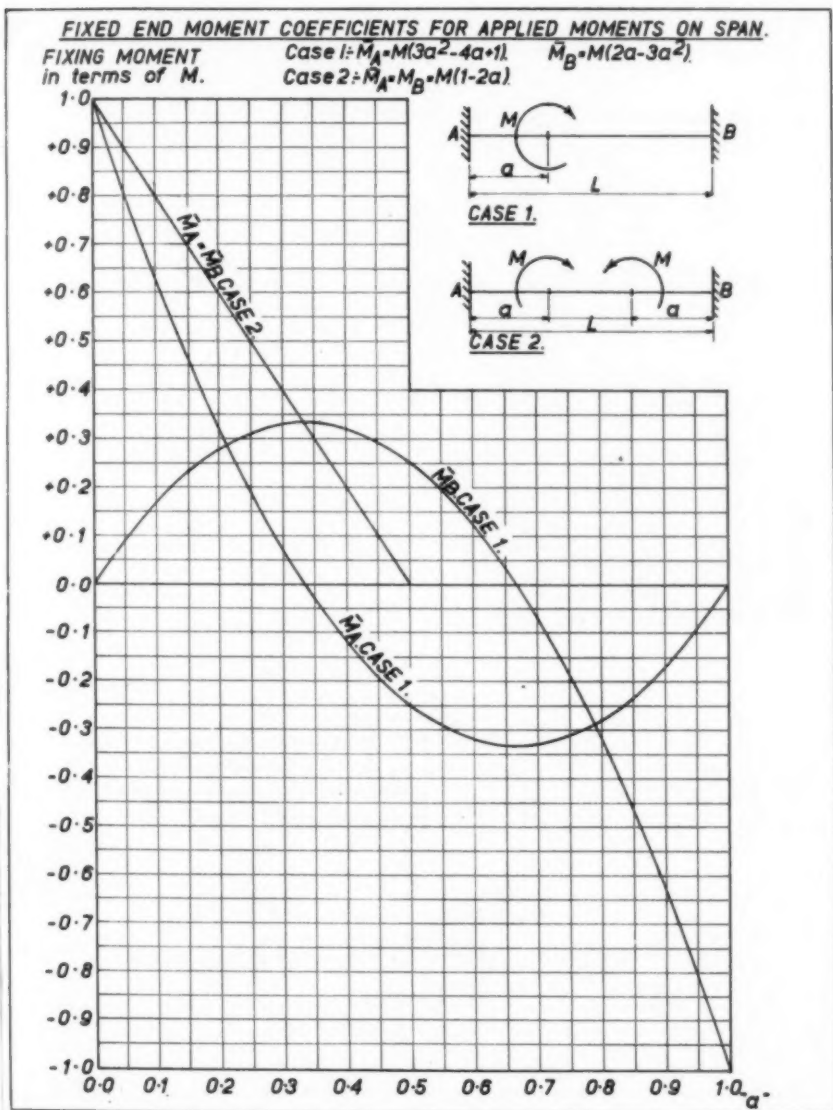


CHART 4.



Examples.

(1) Determine the fixed-end moments caused by the load shown in Fig. 1. $x = \frac{7}{20} = 0.35$. $a = \frac{5}{20} = 0.25$. $b = \frac{8}{20} = 0.40$. Span (L) = 20 ft. Total load (W) = $7 \times 1000 = 7000$ lb. From Chart 1, $\bar{M}_A = 0.133WL = 18,650$ ft.-lb. $\bar{M}_B = 0.101WL = 14,150$ ft.-lb. The effect of concentrated loads may be ascertained by using the curve for $x = 0$ on Charts 1 to 3.

(2) Determine the fixed-end moments caused by the load shown in Fig. 2. The calculations may be set out as in Table I.

TABLE I.

Load No.	Total load (lb.)	x	a	b	\bar{M}_A (ft.-lb.)	\bar{M}_B (ft.-lb.)
(1)	840×15 = 12,600	$15/27$ = 0.555	$4/27$ = 0.148	$8/27$ = 0.296	$0.122WL$ = 41,400	$0.098WL$ = 33,000
(2)	$630 \times 8 \times \frac{1}{2}$ = 2,520	$8/27$ = 0.296	$4/27$ = 0.148	$15/27$ = 0.555	$0.134WL$ = 9130	$0.047WL$ = 3200
(3)	8000	0	$8/27$ = 0.296	$19/27$ = 0.704	$0.147WL$ = 31,700	$0.062WL$ = 13,350
(4)	5500	0	$21/27$ = 0.778	$6/27$ = 0.222	$0.038WL$ = 5700	$0.135WL$ = 20,000
					87,930	69,550

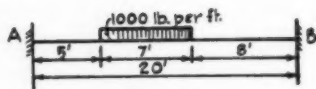


Fig. 1.

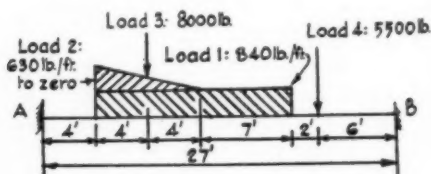


Fig. 2.

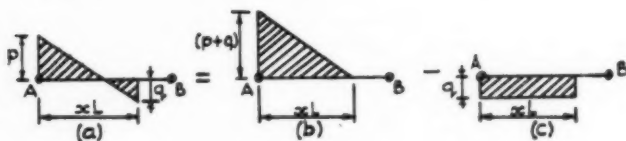


Fig. 3.

In the case of triangular loading it is necessary to ensure that the load slopes in the same direction as that shown on Charts 2 and 3. If this is not the case, the values of a and b must be interchanged. Loading such as that shown in Fig. 3a may be divided as shown in Figs. 3b and 3c. Chart 4 gives the fixed-end moments caused by applied moments.

A Water Tower at Doncaster.

THE tower shown in *Figs. 1 and 2* has recently been built at Cantley, near Doncaster. It has a capacity of 500,000 gallons. Three-quarters of the water supplied to Doncaster is obtained from bore-holes to the east, the level of which rises gradually from about 20 ft. above Ordnance Datum in the east to about 200 ft. above O.D. in the west in a distance of about ten miles.

The tower is intended to provide sufficient storage to ensure continuity of supply if the pumps fail, and a minimum pressure at periods of peak demand which permits more economical operation of the pumping plant.

The tower is on the highest ground between the source of the water and the centre of the city. This position was decided upon because a tank at ground level on the highest ground in the area would be on the opposite side to the sources of water, and the extra cost of mains and pumping would have been greater than the saving in cost afforded by the cheaper tank. As all the pumping plant is in duplicate the reserve storage provided is small.

The bottom water-level is 111 ft. 6 in. above ground level. The foundation is on hard sand 8 ft. 6 in. below ground level and is of beam-and-slab construction. It has a diameter of 83 ft., and a peripheral wall 9 in. wide is provided on the centre-lines of the columns. The twelve columns are on a pitch circle of 69 ft. 7 in. diameter and have an inward taper of 1 deg. This taper is provided also on the external face of the tank and on the pilasters, which form continuations of the columns. The columns are 5 ft. by 5 ft. 6 in. in cross section and have an unsupported length of 105 ft. from the top of the foundation-beam to the soffit of the peripheral tank-beam; the columns end at the floor of the tank.

The wall of the twelve-sided internal shaft is 1 ft. thick, and the dimension across the faces of the shaft is 24 ft. The pumps are supported on the suspended ground floor, and the shaft also supports the pipework and ten flights of reinforced concrete stairs. At the landing of each flight a light open-type metal floor is provided.

The tank has two compartments with

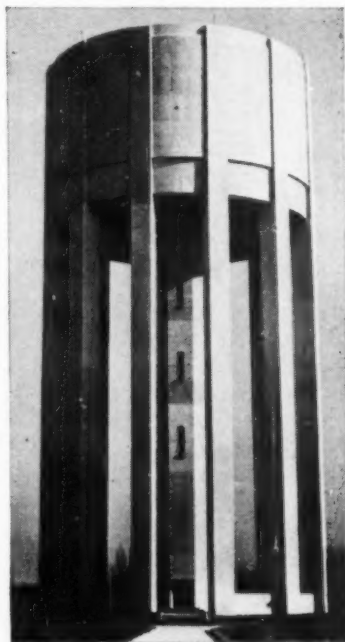


Fig. 1.

equal capacities; the head of water in each is 25 ft. The internal diameter of the external wall is 66 ft. 9 in. and of the internal wall 46 ft. The thickness of the external wall was designed to be 1 ft. 9 in. at the bottom and 1 ft. at the top, and of the internal wall 1 ft. 6 in. at the bottom and 1 ft. at the top. At the request of the contractor, however, the faces of the walls (with the exception of the external face) were made vertical. Both walls were designed as hoops, the external pilasters being very slightly reinforced to avoid cracking due to restraint. The tanks are not lined. Access to the tanks and to the automatic operating valves is obtained from the roof; there is a central shaft of 6 ft. diameter which cantilevers from the floor of the tank and provides access to the roof by a ladder from the tenth landing.

The inlet, outlet, and overflow pipes pass between the beams supporting the bottom of the tank and enter the tank some distance from the central shaft.

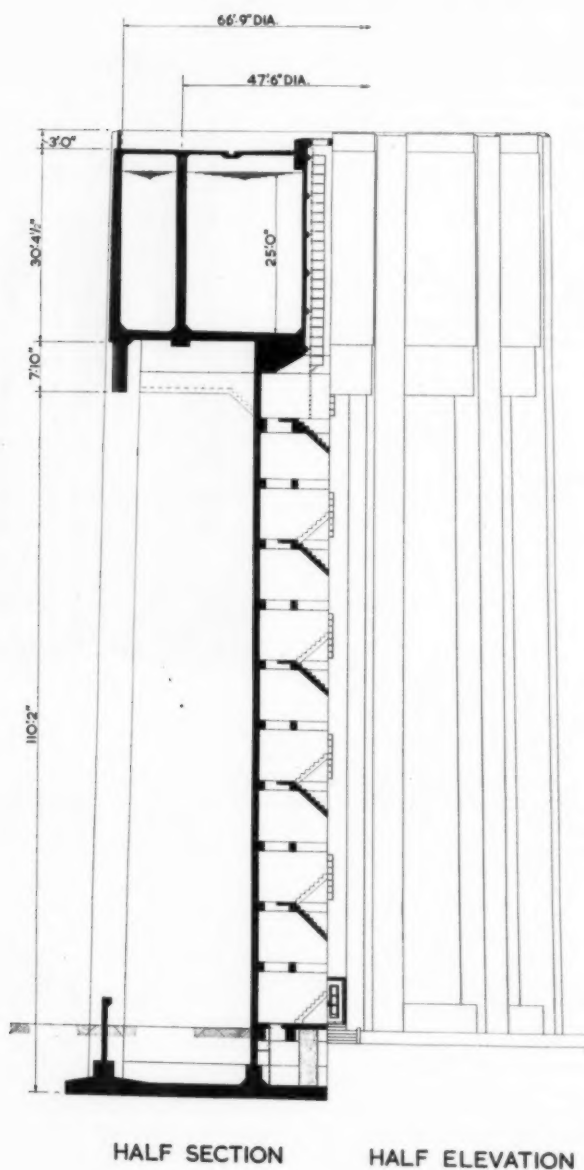


Fig. 2.

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The beams adjacent to these pipes support sub-floors on which the pipes are carried, thus allowing easy access and maintenance in inclement weather.

The beam-and-slab roof is surrounded by a parapet 3 ft. high; drainage for rainwater is provided. The parapet of the tower is 135 ft. above the ground.

Proportions and Testing of Concrete.

The specification required proportions by weight of 1 cwt. of cement to $4\frac{1}{2}$ cwt. of combined dry aggregates. The concrete was compacted by poker vibrators. Tests showed that 74 per cent. of coarse aggregate with a maximum size of $\frac{3}{4}$ in. and 26 per cent. of sand gave a combined grading curve which agreed very closely with Curve No. 1 of D.S.I.R. Road Note No. 4. These proportions were used throughout the work.

Works cube tests were made whilst excavation was proceeding so that the results would be available before concreting started. The coarse aggregate was crushed gravel which was generally smooth and hard; workable concrete was obtained with a water-cement ratio of 0.33.

The works cubes had strengths of 5425 lb. per square inch at seven days. As a means of keeping the water content consistent the grout for coating the face of existing concrete was mixed before the first batch of concrete each day, thereby depositing a film of grout in the drum. Also, the aggregate-hoppers were completely emptied each day to avoid change in the water content due to drainage

during the night. The water in the aggregates was checked each day and sieve analyses were prepared for all aggregates delivered to the site. The variation in grading was found to be between 2 and 3 per cent.

As the work proceeded and the concrete was placed by means of a hoist, hopper, and barrow, the water-cement ratio was increased to 0.37 to increase the workability of the concrete. This change was made when the columns and central shaft were about 20 ft. high.

The first forty cubes, made of concrete with a water-cement ratio of 0.33 and an average slump of $\frac{3}{4}$ in., were cured in water and tested at seven days. Their average weight was 153.87 lb. per cubic foot and the average crushing strength was 5832 lb. per square inch. The standard deviation was 600 lb. per square inch. The remainder of the cubes (72 in number) made of concrete with a water-cement ratio of 0.37 and an average slump of $1\frac{1}{4}$ in. were cured in sand and tested at seven days. Their average weight was 153.56 lb. per cubic foot and the average strength 4998 lb. per square inch. The standard deviation was 637 lb. per square inch. All the cubes were compacted by hand.

Mr. M. Cawley is the Borough Surveyor, Water Engineer, and Planning Officer of the County Borough of Doncaster. The consulting engineers were Messrs. L. G. Mouchel & Partners, and the contractors were the Yorkshire Hennebique Contracting Co., Ltd. For the foregoing information we are indebted to Mr. W. J. Henderson, A.M.I.C.E.

The Volume of Construction.

FIGURES issued by the Ministry of Works show that the total value of contracts for building and civil engineering work placed with contractors in the first three months of this year was £339,000,000, compared with £299,000,000 in the fourth quarter, £341,000,000 in the third quarter, £331,000,000 in the second quarter, and £368,000,000 in the first quarter of 1957. The largest single increase was for work other than house building for public authorities, which increased by £28,000,000 compared with the fourth quarter of 1957, and by £13,000,000 compared with the first quarter of last year.

September, 1958.

Collapse of a Bridge in Canada.

Two of the members of the Royal Commission set up in Canada to inquire into the collapse during construction of the Second Narrows bridge at Vancouver are Mr. Ralph Freeman, C.B.E. (a partner of Messrs. Freeman, Fox & Partners, of London), and Mr. J. R. H. Otter (of Messrs. Rendel, Palmer & Tritton of London).

Restraint of Slabs by Edge-beams.

On page 251 of this journal for July, 1958, the definitions of the symbols ψ and ψ' should read "change in twist of edge-beam per unit length".

A Dam in Italy.

THE Vajont dam, Italy, now in course of construction, is stated to be the highest of its kind in the world. It was designed by Societa Adriatica di Eletticit , of Venice, and is being built by G. Torno & Co., of Milan. The structure will be 872 ft. high, and the maximum water level will be at a height of 864 ft. The length at the top will be 626 ft., and the thickness will vary from 76 ft. at the bottom to 10 ft. at the top. The reservoir will have a capacity of 196,000,000 cu. yd. The gorge in which the dam is being built is shown in Fig. 1. The first operations were the construction of tunnels 6600 ft. long and the building of a reinforced concrete bridge 230 ft. long spanning the gorge.

The rock consists essentially of tough oolitic limestone and liasic siliciferous limestone with thin layers of organic material and hard nodules of radiolaria. The siliceous material reduces the speed of the drills while the thin layers of slime between the strata of limestone often cause the drill steels to jam. The seatings for the abutments are being excavated in two gulfs rising to 890 ft. above the bottom of the gorge, and the drillers are suspended against the face of the cliffs. The benches are sloped so as to allow the fractured rock to fall to the bottom of the gorge.

In order to protect the drillers some 430,000 sq. ft. of wire netting are provided along the side of the gorge, and walkways of perforated iron sheets are fixed to the face of the cliff at vertical intervals of 164 ft.; these are connected by enclosed ladders separated by wooden landings at every 16 ft. of height. The blasting has been done with deep vertical holes of large diameter and shallower holes of smaller diameter in order to avoid shaking the rock, particularly near the seatings of the abutments. Light and heavy rock drills have both been employed, supplemented by 40-lb. drills for scaling the walls of the gorge. All the rock drills are flushed by air instead of water to avoid the risk of freezing during the winter and to allow the use of less powerful explosives. The work is being done in three shifts of eight hours each day and is floodlit at night. The total volume of blasted rock is 510,000 cu. yd.



Fig. 1.

Owing to the narrow width of the bottom of the gorge most of the excavated material is hauled through a tunnel at one side. A belt-conveyor at the upstream entrance of the tunnel loads the rock on to trucks of 8 tons capacity, which pass through the tunnel and then along a road which was made by filling the bed of the river with rock.

The compressed air is supplied by two Atlas Copco reciprocating compressors with a capacity of 1000 cu. ft. per minute. Four air containers of 106 cu. ft. capacity are installed outside the compressor house, and the distribution line is more than three miles long. The rock near the abutments has been consolidated by injecting cement; holes up to 100 ft. long have been drilled into the rock and filled with cement grout under pressure.

Losses of Prestressing Force.*

By PAUL W. ABELES, D.Sc. (Vienna), M.I.Struct.E.

Losses with Pre-tensioning.

It is normal practice to specify the pre-tensioning force required, from which the initial stress in the steel p_i is determined. The stress p_t in the steel when the prestress is applied to the concrete is obtained from $p_t = p_i - Lp_i$, in which Lp_i represents the losses occurring at transfer.

In order to determine the maximum possible stress in the concrete and the minimum effective prestress, it is necessary to allow for minimum losses before prestressing and maximum losses after. Consequently it is advisable to assume that no losses due to shrinkage occur before prestressing, the only loss considered

TABLE III.—RATIOS OF LOSSES DUE TO CREEP.

$f_{s.av.}$	TIME AFTER TRANSFER DURING WHICH M_T ACTS ALONE							
$0.9f_{tk}$	1 WK.	2 WKS	3 WKS	1 MTH.	2 MTHS	3 MTHS	6 MTHS	1 YR.
0	0.25	0.35	0.43	0.50	0.65	0.75	0.85	0.95
0.1	0.325	0.415	0.487	0.55	0.685	0.775	0.865	0.955
0.2	0.40	0.48	0.544	0.60	0.72	0.80	0.88	0.96
0.3	0.475	0.545	0.601	0.65	0.755	0.825	0.895	0.965
0.4	0.55	0.61	0.658	0.70	0.79	0.85	0.91	0.97
0.5	0.625	0.675	0.715	0.75	0.825	0.875	0.925	0.975
0.6	0.70	0.74	0.772	0.80	0.86	0.90	0.94	0.98
0.7	0.775	0.805	0.829	0.85	0.895	0.925	0.955	0.985
0.8	0.85	0.87	0.886	0.90	0.93	0.95	0.97	0.99
0.9	0.925	0.935	0.943	0.95	0.965	0.975	0.985	0.995
1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

being that due to elastic shortening Lp_e , and therefore for maximum stress in the concrete $Lp_t = Lp_e$ and $p_t = p_i - Lp_e$. No loss due to relaxation of the steel has been allowed for; however, the relaxation depends to some extent on the time which elapses between tensioning and transfer, and if the rate at which losses due to relaxation occur is known a suitable allowance may be made. The minimum effective prestress p_e is given by

$$p_e = p_i - Lp_s - Lp_c - Lp_r = R_o \cdot p_i = R_e \cdot p_t,$$

in which Lp_s is the loss due to shrinkage, Lp_c the loss due to creep, and Lp_r the loss due to relaxation of the steel.

The reduction factors R_o and R_e vary considerably, being dependent mainly on the stress in the concrete adjacent to the steel. It should be noted that this stress may be much less after the loading is applied than at transfer, and allowance may be made for this when calculating the losses due to creep if the length of time which will elapse, before the full dead load is applied, is known.

* Continued from August, 1958.

CHART I.

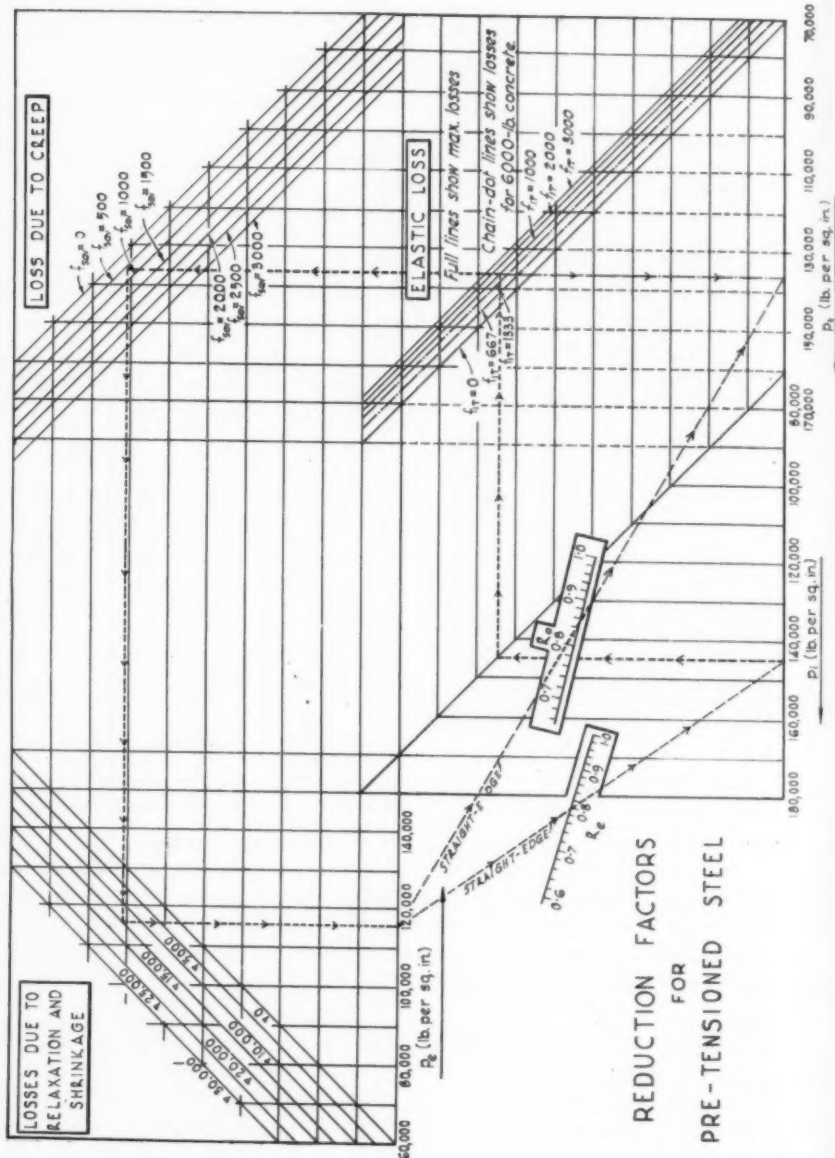


Table III gives the equivalent stress which may be used for this purpose, for various values of initial stresses and stresses due to dead load, and for various periods of time, when the stress in the concrete at transfer is $0.9f_{st}$.

The evaluation of the reduction factors R_o and R_e is facilitated by Chart No. 1. The chart is entered on the scale marked p_i and, after losses due to elastic shortening are taken into account, p_t may be determined as shown by the chain-dotted line. Losses due to creep, relaxation, and shrinkage are next allowed for, the value of p_e being then determined. A straight-edge laid between p_e and p_i and p_t and p_e enables R_o and R_e to be determined on the nomograms. In the example shown on the chart, p_i is assumed to be 145,000 lb. per square inch and f_{1T} is assumed to be 1000 lb. per square inch; the value of p_t is found to be 136,000 lb. per square inch. Losses due to creep are determined assuming $f_{s.av.}$ to be 600 lb. per square inch. Losses due to shrinkage and relaxation are assumed in this example to be 15,000 lb. per square inch; p_e is found to be 116,000 lb. per square inch and R_e and R_o are found to be 0.795 and 0.84 respectively.

If a long period of time is likely to elapse before the full dead load is applied to the member, the maximum losses due to creep should be taken into account, as shown by the full lines on the chart. If, however, it is known that the full dead load will be applied soon after transfer, and that $f_{s.av.}$ will be appreciably less than $0.9 f_{1T}$, the value of $f_{s.av.}$ may be taken from Table III, and the losses determined from the lines shown in chain-dot on the chart. Losses indicated by the chart are those recommended in the British draft Code.

Losses with Post-tensioning.

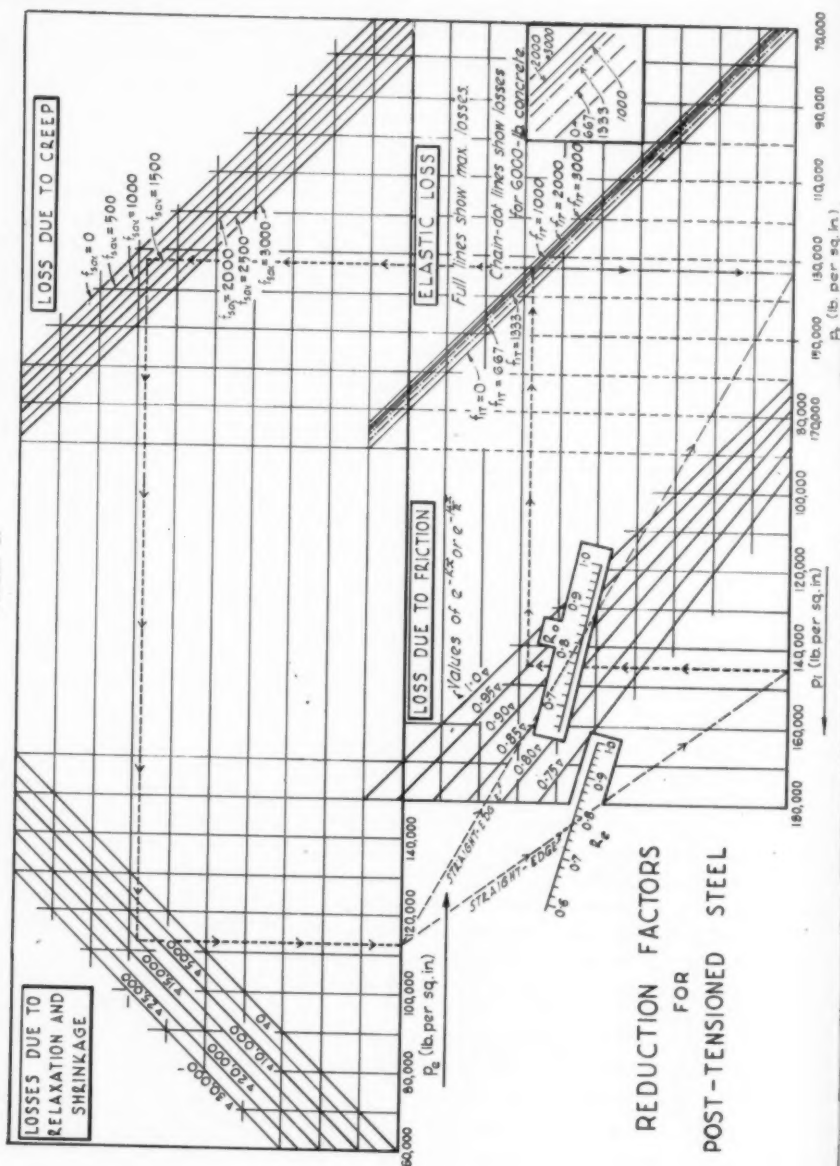
The conditions assumed in computing the losses with post-tensioned steel are identical in principle with those for pre-tensioning, but different values are employed. Losses occurring at transfer include those due to friction (Lp_f) in addition to those due to elastic shortening (Lp_e) where applicable: $p_t = p_i - Lp_e - Lp_f$. The calculated elongation will agree with that actually observed only when losses due to friction are allowed for accurately. The common practice of increasing the initial prestressing force, regardless of its magnitude, until the calculated elongation is obtained may cause dangerous local compressive stresses, and the designer should clearly specify whether a definite elongation is required or whether losses due to friction have been allowed for; in the latter case a definite prestressing force and an expected elongation should be specified. The first method may cause an appreciable increase in the initial tensile stress in the steel, and it is necessary to ensure that the permissible value is not exceeded. If the full elongation is obtained when losses due to friction have been allowed for, then a higher effective prestress than that assumed in the calculations may occur, the extent of which will depend entirely on the variation of stress along the steel due to frictional losses where the slope changes.

The effective prestress p_e is given, as before, by

$$p_e = p_i - Lp_e - Lp_c - Lp_r = R_o \cdot p_i$$

and the same considerations apply. Shrinkage and creep may be less in view of the greater strength of the concrete when the prestress is applied, and p_t may be less when losses due to friction are large. Chart No. 2 reduces the work involved in evaluating the expression. The chart is similar to that for pre-tensioned steel

CHART 2.



but also includes curves for frictional losses. In the example shown on the chart, the value of p_i is 145,000 lb. per square inch, and losses due to friction (assuming e^{-Kx} to be 0.95) and elastic shortening (assuming $f_{1T} = 1000$ lb. per square inch) are obtained. The value of p_e is found to be 133,000 lb. per square inch. Losses due to creep are determined, the value of $f_{s.av.}$ being assumed to be 600 lb. per square inch, and the losses due to shrinkage and relaxation are assumed to be 15,000 lb. per square inch. p_e is found to be 112,500 lb. per square inch, R_e is 0.775 and R_o is 0.84.

The assessment of the losses that arise when prestressing is done in two or more stages is slightly more complicated, and is not considered here.

Examples.

The following examples illustrate the method of computing losses occurring with pre-tensioned and post-tensioned steel for various values of prestress in the concrete, making certain assumptions concerning the age of the concrete when the prestress is applied and the stresses due to the dead load at the time

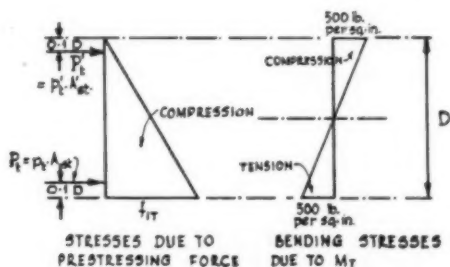


Fig. 6.—Stress Diagram for Example I.

of prestressing. Similar calculations can be made for any other value of prestress, and the corresponding reduction factor R_o obtained.

EXAMPLE I. PRE-TENSIONED STEEL.—A symmetrical precast beam with pre-tensioned wires is considered, in which the application of the prestressing force gives rise to a triangular distribution of stress, and the maximum bending stresses due to the weight of the beam are ± 500 lb. per square inch. It is also assumed that the centroid of the tensile steel is at a distance 0.1D from the bottom, the stress in the concrete in the plane of the centroid amounting therefore to $0.9 f_{1T}$ (Fig. 6). If the modular ratio corresponding to the strength of the concrete at the time of prestressing is 5, then the losses due to elastic shortening are $4.5 f_{1T}$.

Young's modulus for the wire is assumed to be 29×10^6 lb. per square inch, and all the shrinkage of the concrete is assumed to occur after it is prestressed, the concrete being moist cured until it is prestressed. Losses due to shrinkage are therefore 0.03 per cent., or 8700 lb. per square inch. At midspan the stresses due to the weight of the beam oppose those due to the prestressing force. The initial stress is therefore reduced from f_{1T} to $f_{1T} - 500$, and the stress in the plane of the centroid of the tensile steel is between 85 per cent. and 95 per cent. of the maximum stress at the bottom face.

It is assumed that the beam may not be loaded further for a considerable time,

and that most of the creep will therefore take place with this distribution of stress; however, since shrinkage of the concrete and relaxation of the steel will also occur during this period, and the creep will occur gradually, it is sufficient to allow for an average creep corresponding to 90 per cent. of the stress at the centroid of the steel. If this stress is assumed to be f_{st} , the losses due to creep are $0.33 \times 10^{-6} \times 29 \times 10^6 \times 0.9 = 8.62 f_{st}$, and are evaluated in Table IV for stresses at transfer of 1500, 2000, 2500, and 3000 lb. per square inch. In every case a maximum relaxation of 10,000 lb. per square inch in the steel has been taken into account. A graph showing the reduction factors R_o and R_e (the latter

TABLE IV.—CALCULATIONS FOR EXAMPLE I.

$$P_i = 157,000 \text{ lb. per sq. in.} \quad L_{p_g} = 4.5 f_{i_T} \quad L_{p_c} = 8.62 f_{st} \\ L_{p_g} = 8700 \text{ lb. per sq. in.} \quad L_{p_r} = 10,000 \text{ lb. per sq. in.} \quad f_{i_T} = 500 \quad f_{st} = +500$$

f_{i_T}	L_{p_c}	P_t	f_{it}	f_{st}	L_{p_c}	$L_{p_g} + L_{p_c} + L_{p_r}$	P_e	$R_{0.00}$	R_e	$R'_{0.00}$	R'_e	$R_{0.28}$
1500	6750	150,250	1000	950	8200	26,900	123,950	0.821	0.786	0.850	0.846	0.934
2000	9000	148,000	1500	1400	12,100	30,800	117,200	0.792	0.747	0.846	0.841	0.920
2500	11,250	145,750	2000	1850	15,900	34,600	111,150	0.763	0.710	0.844	0.837	0.906
3000	13,500	143,500	2500	2300	19,800	38,500	105,000	0.732	0.670	0.841	0.833	0.891

$$R_{0.00} = \frac{P_e}{P_i} \quad R_e = \frac{P_e}{P_t} \quad R'_{0.00} = \frac{P'_e}{P'_t} \quad R'_e = \frac{P'_e}{P'_t} \quad R_{0.28} = \frac{P_{e,28}}{P_i}$$

All stresses given in lb. per sq. in.

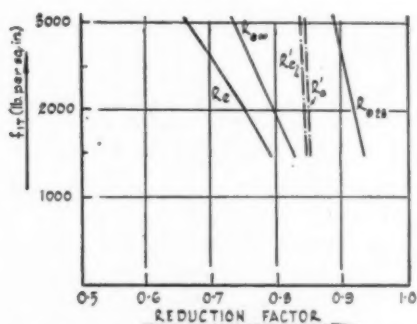


Fig. 7.—Reduction Factors for Example I.

indicating the entire losses between the initial prestress and the minimum possible effective prestress) is also given, based on an initial tensioning stress of 70 tons per square inch (157,000 lb. per square inch—Fig. 7). Losses and reduction-factors for the steel near the upper face of the beam may be calculated in the same way and values of the reduction-factors R'_o and R'_e for this steel are given in the table and on the graph. In certain circumstances—for example, for acceptance tests—it may be desired to calculate the losses after a certain period; in this case the full elastic loss should be allowed for, and the combined values of the total losses due to shrinkage, creep, and relaxation may be multiplied by the factor given in Fig. 8, although the time at which the losses due to relaxation occur is not known with any accuracy. Fig. 8 is in accordance with Table III,

which relates only to creep of the concrete ; it is therefore based on the assumption that the losses due to shrinkage and relaxation occur at the same rate as those due to creep, and R_o is assumed to represent the losses after two years. Generally R_e and R_o indicate the reduction of the initial prestress and that occurring at transfer respectively, and both give the same value for the minimum effective prestress that may occur after a long period of time : $p_e = R_{o\infty} p_t = R_e p_t$. When additional reduction-factors are required, R_o is termed $R_{o\infty}$ to distinguish it from R_{ot} or R_{o28} ; similarly $R_{e\infty}$, R_{et} , and R_{e28} must be introduced when

TABLE V.—REDUCTION OF LOSSES DUE TO CREEP.

$$f_{s,av} = \frac{f_{st}}{3} + \left(\frac{2}{3} \times 750 \right) \quad Lp_c = 8.61 f_{s,av}$$

f_{it}	$f_{s,av}$	Lp_c	ΔLp_c	p_e	R_o	R_e
1500	$500 + 317 = 817$	7030	1170	124,420	0.827	0.792
2000	$500 + 467 = 967$	8320	3780	120,980	0.817	0.770
2500	$500 + 617 = 1117$	9620	6380	117,530	0.806	0.748
3000	$500 + 767 = 1267$	10,910	8890	113,690	0.793	0.726

All stresses in lb. per sq. in.

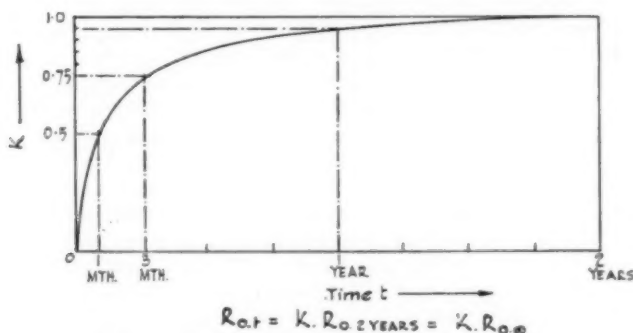


Fig. 8.—Relation between Losses and Time.

specific times are considered. When R_e and R_o are used without further subscripts, they represent $R_{e\infty}$ and $R_{o\infty}$.

Many variations are, of course, possible in the value of the initial tensioning stress, the distribution of the prestress, the stresses due to the weight of the beam, and the age of the beam when it is first loaded. If the beam be loaded at an early age, for example, losses due to creep will be smaller, and it may perhaps be assumed that 50 per cent. of these losses occur before loading and the remainder after loading when the stress causing the creep is much less ; alternatively, the values given in Table V may be used. The data in Table V are calculated on the assumption that the creep which occurs when the stress in the concrete adjacent to the steel is high is one-third of the total creep, and that the stress after the member is placed in position is 750 lb. per square inch. The equivalent stress upon which the calculation of the loss due to creep is based is then

$$f_{s,av} = \frac{1}{3} f_{st} + \frac{2}{3} \times 750 = \frac{1}{3} f_{st} + 500 \text{ lb. per square inch.}$$

In the method described in the foregoing an attempt is made to ascertain the maximum possible losses in order to determine the minimum effective prestress that can reasonably be expected to occur, and to ensure that the beam will behave as predicted by the calculations. These considerations also require that the strength of the concrete when it is prestressed be at least 2.5 times the maximum stress in the concrete at transfer, although the British draft Code recommends a strength of only twice the stress at transfer.

An alternative method sometimes adopted is to ignore the stress in the concrete at transfer, which may be greater than that permitted by the draft Code—for example, two-thirds of the strength—and to simplify the calculation in such a way as to allow for as little loss as possible. Losses due to elasticity and creep, for example, are calculated for the stress in the concrete adjacent to the centroid of the entire steel, instead of the centroid of the steel in the tensile zone. Losses due to creep are calculated on the assumption that the member is immediately loaded, although in fact it may be previously exposed for some time in the open air without curing. Further, if the margin between the compressive stress and the strength at the time of application of the prestress is small, the actual creep is much greater than that assumed, since creep is proportional to stress only when the stress is less than one-third of the strength.* It is therefore reasonable to conclude that the effective prestress remaining in the concrete will be much less than that assumed in the calculations, and the factor of safety against cracking will be greatly reduced or even non-existent. If the steel is well distributed throughout the tensile zone this will not be particularly disadvantageous, since high tensile stresses in the concrete, or even fine hair cracks that may occur under working load, will usually be harmless, but it is the writer's opinion that the purpose of a calculation is to obtain results which agree as closely as possible with the actual behaviour; moreover, the application of such a method of calculation to members in which the steel is not so well distributed and the bond is not so efficient—as may well be the case with post-tensioning—may be dangerous.

EXAMPLE II. POST-TENSIONED STEEL.—Consider a symmetrical member with post-tensioned steel, part of which is bent up so that the stresses due to the weight of the beam immediately counteract those due to the prestressing force, if measures are taken to ensure the destruction of any adhesion between the soffit of the beam and the shutter. As in Example I, it is assumed that the distribution of stress due to the prestress alone is triangular, the prestress at the top face being zero and that at the bottom being f_{1t} at the time of prestressing, and that the maximum bending stresses due to the weight of the beam are ± 500 lb. per square inch. The resultant stresses at transfer are therefore $f_{1t} = f_{1T} - 500$, and $f_{2t} = 500$ lb. per square inch. If the tensioning force is applied simultaneously to all the steel then there is no loss due to elastic shortening, but if the steel members are tensioned one after another some elastic shortening will occur. It is assumed that the net initial tensioning stress is the same for all the steel members, except for the differences in losses due to friction; the average loss due to elastic compression will then not exceed half the loss that occurs with pre-tensioning. Thus for $m = 5$ the maximum loss due to elastic shortening will be $2.5f_{st}$, in

* This figure is based on tests by Dr. W. H. Glanville, as described in reference⁽¹⁾. Non-linear creep has recently been shown to occur at stress/strength ratios in excess of 0.25 for a cylinder strength of about 6500 lb. per square inch (A. M. Freudenthal, "Creep Effect in the Analysis of R.C. Structures". Preliminary Report of 5th Congress, I.A.B.S.E., Lisbon, 1956). This means that proportionality between creep and stress may be limited to a stress as small as $0.75 \times 0.25 = 0.19$ of the cube strength, for a ratio of cylinder strength to cube strength of 0.75.

which f_{st} is the resultant stress in the concrete in the plane of the centroid of the steel. Table VI and Fig. 9 show losses for various values of prestress for an initial net tensioning stress of 65 tons per square inch (145,600 lb. per square inch). This reduced stress allows a margin for frictional losses.

Losses due to shrinkage are assumed to be the same as those in Example I, although two-thirds of this value would be permissible. However, such a reduction is based on the assumption that some shrinkage has occurred before

TABLE VI.—CALCULATIONS FOR EXAMPLE II.

$p_i = 145,600 \text{ lb. per sq. in.}$ $L_{pe} = 2.5 f_{st}$ $L_{ps} = 4 f_{st}$
 $L_{ps} = 6700 \text{ lb. per sq. in.}$ $L_{pr} = 10,000 \text{ lb. per sq. in.}$ $f_{re} = f_{ir} - 500$ $f_{rs} = +500$

f_{ie}	f_{st}	L_{pe}	p_t	L_{pc}	$L_{ps} + L_{pr}$	p_e	$R_{0.00}$	R_e	$R_{0.25}$
1000	950	2400	143,200	3800	22,500	120,700	0.843	0.829	0.926
1500	1400	3500	142,100	5600	24,300	117,800	0.829	0.809	0.915
2000	1850	4600	141,000	7400	26,100	114,900	0.815	0.790	0.900
2500	2300	5800	139,800	9200	27,900	111,900	0.800	0.769	0.883
3000	2750	6900	138,700	11,000	29,700	109,000	0.786	0.750	0.868

All stresses given in lb. per sq. in.

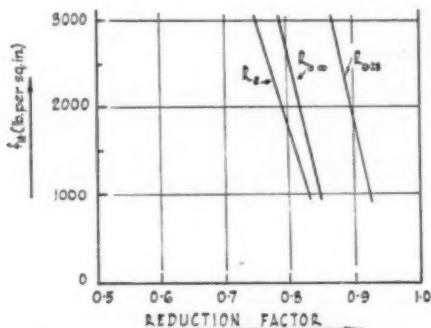


Fig. 9.—Reduction Factors for Example II.

prestressing, in which case cracks may have occurred, and an assumed factor of safety against cracking does not apply; it is therefore wiser, as already stated, to consider the entire loss due to shrinkage but to insist on reliable moist curing of the concrete until it is prestressed.

The losses due to creep are reduced from 0.33×10^{-6} to 0.25×10^{-6} , on the assumption that the age of the concrete when it is compressed is considerably greater than is the case in the pre-tensioning process, and also because members with post-tensioned steel are usually loaded shortly after prestressing, and only a small part of the creep will take place under the stress f_{st} , the remainder occurring with a much smaller stress $f_{s.av.}$. The actual loss due to creep may therefore be between 0.5 and 0.75 of that considered in Example I, that is between 0.5×0.75 and 0.75×0.75 of $8.62 f_{st}$, which amounts to $3.23 f_{st}$ to $4.83 f_{st}$. In Table VI a value of $4 f_{st}$ is allowed, together with a loss of 10,000 lb. per square inch due to relaxation of the steel.

Here again many variations are possible, but Table VI and the graphs indicate the manner in which the reduction factors R_e and R_o may be represented. Values

of $R_{0.28}$ are also included, calculated on the same basis as for Example I. Numerous similar graphs and tables can easily be prepared so that the values of R_0 for any given requirements are immediately obtained.

Smaller losses due to relaxation may also be considered, provided that it is certain that such losses will occur with the steel used.

In the case of high-alloy bars, the factors R_0 are completely different. On the one hand the initial stress in the steel is much less, and on the other hand, so far as is known, no losses due to relaxation need be considered. Despite the latter advantage, however, the comparative reduction factors may be smaller in view of the lower initial stress in the steel, of which the losses obviously represent a greater proportion.

An Elevated Motorway.

THE proposed motorway, nearly twelve miles long, leading from the western outskirts of London to Slough, will form the first stage of the projected London-South Wales road and will also, by means of a spur road, provide a quicker route from west London to London Airport.

The first part of the motorway will be built over the Great West Road. The present proposal (which is subject to alteration) is that a single line of supporting piers (Fig. 1) will be built between the existing dual carriageways, to avoid interference with traffic on the lower road. The elevated part of the motorway will be about a mile in length, and about 25 ft. above the existing road; it will then rise to a height of about 65 ft. to pass over a factory, and descend afterwards

towards ground level. The total length of the viaduct will be $1\frac{1}{2}$ miles. The remainder of the motorway will be at or near the level of the ground.

The motorway and the spur road to the airport will have dual carriageways. Five bridges and link roads will be built to provide access to the motorway from existing and proposed major roads and from the spur road to the airport. Of the other twenty-four bridges which will be required, ten will be over waterways, three over or under railways, and eleven over or under secondary roads.

The viaduct will be designed for the full standard Ministry of Transport loading. The cost of the scheme is estimated to be about £12,000,000. Sir Alexander Gibb & Partners are the consultants.

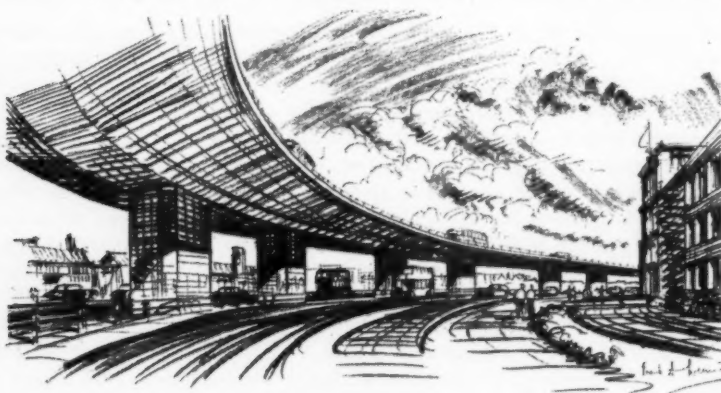


Fig. 1.

An Investigation of Prestressed Bridges.

A DETAILED study of the processes connected with the erection of prestressed concrete bridges has been commenced by Concrete, Ltd., at their works at Hounslow, Middlesex. Five types of structures are being investigated, and the operations studied include the erection of precast members, methods of shuttering for bridge decks and stiffeners, transverse prestress-

ing and reinforcement, and the production of beams with post-tensioned steel. The objects of the work are the determination of the best methods of carrying out the more intricate processes, the determination of the times required for estimating and for the control of costs, and the training of operating teams. It is hoped to obtain information on the relative costs



Fig. 1.—View of Demonstration Area.

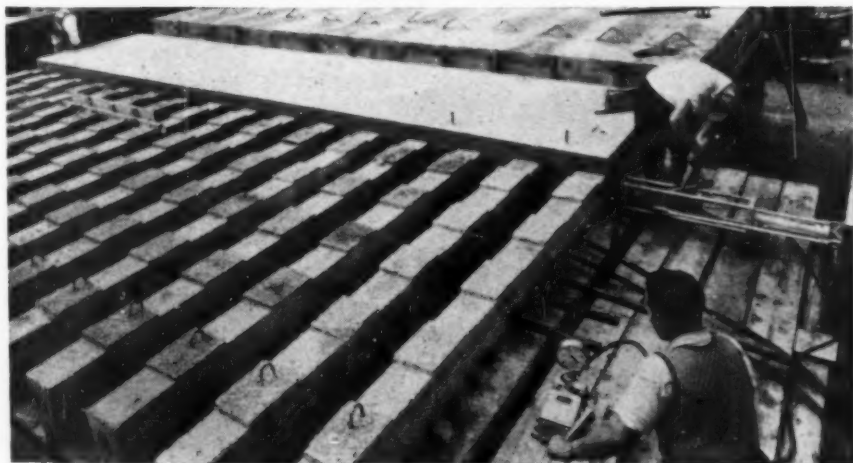


Fig. 2.—Transverse Prestressing of Deck.

of bridges built according to most of the present methods of design, with pre-tensioned and post-tensioned steel. The data obtained will be made available to consulting engineers and contractors.

During a recent demonstration of the work investigations of the following procedures were in progress.

(1) The erection of six hollow bridge beams (*Fig. 1*), 54 ft. long and each weighing $9\frac{1}{2}$ tons, including the use of rubber bearings and methods of aligning the holes for transverse cables.

(2) The erection of a composite bridge (*Figs. 2 and 3a*) suitable for short spans, comprising precast inverted T-beams

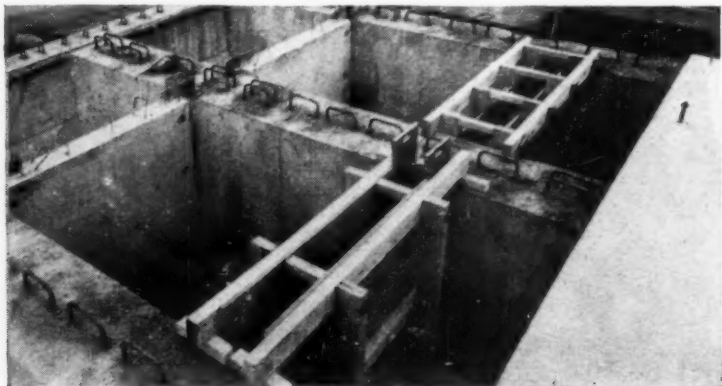


Fig. 3.—Stiffening Slabs.

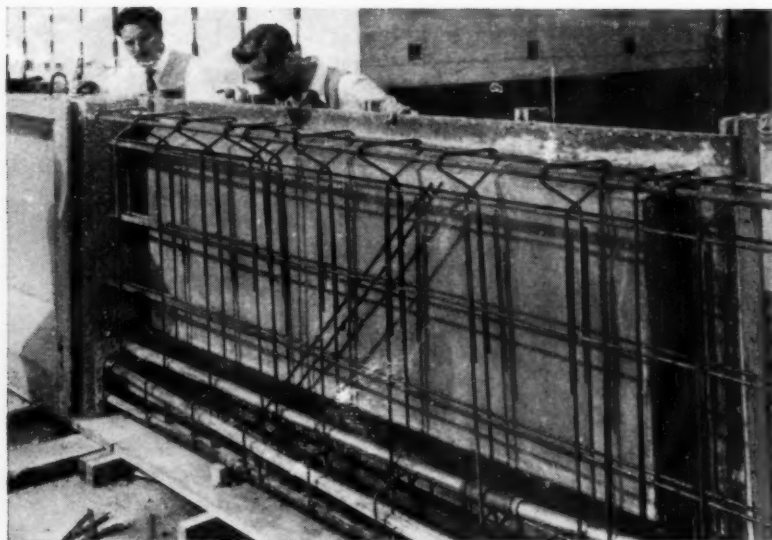


Fig. 4.—Mild Steel Reinforcement and Ducts for Prestressing Cables for Beam 54 ft. Long.

with pre-tensioned steel and infilling concrete cast in place. The transverse reinforcement in one half of the bridge is of mild steel; the other half is transversely prestressed by the Magnel-Blaton system.

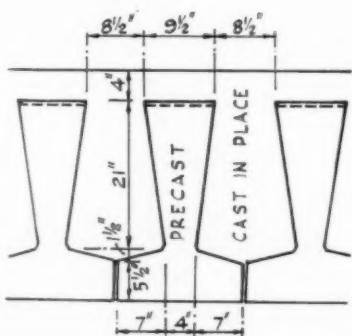
(3) The erection of a composite bridge (Fig. 5b) suitable for medium spans, comprising precast inverted T-beams with wide flanges and pre-tensioned steel and

a deck slab cast in place with permanent shuttering. Both precast and cast-in-place stiffeners are being used, and the operations necessary for transverse prestressing by the Gifford-Udall-CCL system are being studied.

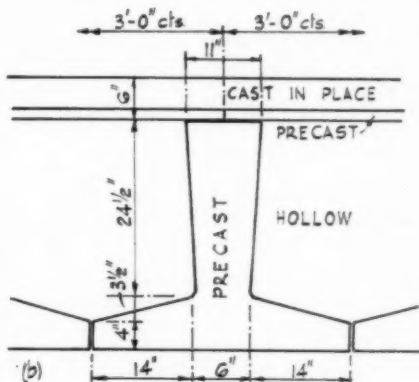
(4) The erection of a bridge (Fig. 5d) comprising precast hollow rectangular beams (with pre-tensioned steel) placed

TABLE I.

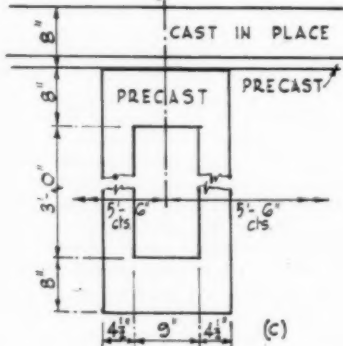
DESIGN SHOWN IN	FIG 5a	FIG 5b	FIG 5c	FIG 5d
DETAILS OF CONCRETE CAST IN PLACE:	1 1/8" BETWEEN AND OVER PRECAST UNITS	R.C. DECK SLABS R.C. STIFFENERS AT 8'-0" CTS	R.C. DECK SLABS R.C. STIFFENERS AT 8'-0" CTS	CONCRETE WITH 1-in. AGGREGATE IN JOINTS
WEIGHT OF PRECAST BEAM:	6-25 TONS	11-9 TONS	14-5 TONS	10-9 TONS
PRE-TENSIONED WIRES:	61 W# 0-2" DIA	135 W# 0-2" DIA	148 W# 0-2" DIA	97 W# 0-2" DIA
STIRRUPS:	10 W# 3/8" DIA. 30 W# 3/8" DIA	24 W# 3/8" DIA. 50 W# 3/8" DIA	45 W# 3/8" DIA	35 W# 3/8" DIA
RESISTANCE TO SHEAR:	CASTELLATIONS 12" LONG, 1" DEEP	86 W# 3/8" DIA BARS	130 W# 3/8" DIA BARS	NO SPECIAL PROVISION
REINFORCEMENT TO CONCRETE CAST IN PLACE	1/8" DIA. AT 2'-0" CTS	3/8" DIA. AT 6" CTS	7/16" DIA. AT 6" CTS	NONE
TRANSVERSE PRESTRESSING	NONE	4/0-276 CABLES AT 8'-0" CTS	4/0-276 CABLES AT 8'-0" CTS	12/0-276 CABLES AT 8'-0" CTS
PRECAST CONCRETE	60-5 cu.ft. 78-0 cu.ft.	78-0 cu.ft. 25-0 cu.ft.	42-5 cu.ft. 38-5 cu.ft.	72-9 cu.ft. 15-9 cu.ft.
PRECAST CONCRETE	41 W#	49 W#	25-6 W#	48-9 W#
0-2" DIA WIRE:	1-40 cwt.	1-96 cwt.	1-99 cwt.	1-30 cwt.
MILD STEEL IN PRECAST CONC	0-78 cwt.	0-35 cwt.	0-41 cwt.	—
MILD STEEL IN SITE CONCRETE	—	20-0 cwt.	20-0 cwt.	—
0-276 DIA WIRE:	—	5 W# 4/0-276	10 W# 4/0-276	10 W# 12/0-276
ANCHORS (TOTAL):	—	—	—	—



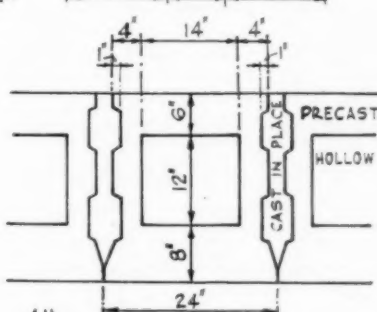
(a)



(b)



(c)



(d)

Fig. 5.—Types of Construction.

side by side. The formation of transverse ducts and the placing and tensioning of transverse steel by the P.S.C. system are being studied.

(5) The erection of a composite bridge (Figs. 3 and 5c) comprising precast hollow rectangular beams with pre-tensioned steel and a deck slab cast in place. The operations being studied include the casting of the stiffeners, the shuttering for the deck slab, and transverse prestressing by the Gifford-Udall-CCL system using stranded cables.

(6) The prestressing of a beam 54 ft.

long (Fig. 4) by the Freyssinet and Lee-McCall systems.

(7) The assembly and jointing of four precast sections of a beam, including the protection of cable ducts.

(8) The grouting of several types of post-tensioned cables and bars.

The design and details of the structures tested are given in Table 1. Each bridge has a span of 50 ft., and the designs are based on the abnormal loading of the Ministry of Transport. It is expected that the programme will be completed in about a year.

THE "AGGREGATE TRANSFER" PROCESS.

THE aggregate-transfer method is a means of embedding selected aggregates in the face of concrete cast in place. Since only a single layer of aggregate is used for the surface the method permits the use of aggregates which would be too expensive if used throughout the concrete. The process originated in the U.S.A., and the following notes are based on trials made in England by the Cement and Concrete Association.

Plywood $\frac{1}{8}$ in. thick is cut to size, and strips of plywood, previously coated with paraffin wax, are pinned to the edges to form a tray. The depth of the tray is equal to the thickness of the largest aggregate to be used. In the tray, a mixture of an adhesive and filler is spread to a depth equal to half the nominal size of the aggregate. The aggregate is then sprinkled over the surface and pressed firmly into the matrix with a float faced with sponge rubber. When the matrix reaches the top of the tray the tray is tilted to allow surplus aggregate to fall off. Any gaps are made good, and the prepared panel is left at least four days for a water-soluble cellulose adhesive to harden, or a shorter period if a stronger adhesive is used.

The prepared panels, which may be of any size up to the standard size of plywood (8 ft. by 4 ft.), are placed within the shutters to which they are fixed by staples. The sides of the trays are left in position as long as possible to prevent the aggregate from being dislodged. When these sides are removed the joints between the panels are filled with strips of caulking compound. If the panels are pushed firmly against those already in place, a well-filled joint of about $\frac{1}{8}$ in.

will be formed.

The structural concrete should have a slump of from 3 in. to 4 in. In the U.S.A. air-entrained concrete is preferred, but the Association states that this does not seem to be necessary. Compaction should be with an internal vibrator, care being taken to keep the vibrator from the panel.

When the concrete has hardened the shutters are removed, leaving the plywood bottoms of the trays in position. These are then removed, and the aggregate remains embedded in the face of the concrete. Any adhesive covering the face is removed by washing and scrubbing.

ADHESIVES AND FILLERS.—In the U.S.A. it is the practice to use a water-resistant adhesive such as a mixture of nitro-cellulose, dammer gum, and acetate. In the work done by the Cement and Concrete Association an animal glue that just gels at 15 deg. C. and a water-soluble cellulose have been tried, and both have been successful. The adhesive must be strong enough to secure the aggregate in place while the structural concrete is placed, but not so strong that the plywood cannot be removed without damage.

Sand which passes a 14-mesh sieve has been found to be the most suitable filler. It does not shrink (as does sawdust, which has also been tried), and the layer of sand adhering to the concrete between the particles of aggregate, if chosen for its colour, adds to the final appearance.

AGGREGATES.—Aggregates should be as nearly as possible of single size. Gradings of $\frac{3}{8}$ in. to $\frac{1}{4}$ in., $\frac{1}{4}$ in. to $\frac{1}{8}$ in., $\frac{3}{8}$ in. to $\frac{1}{2}$ in., and $\frac{1}{2}$ in. to $\frac{3}{4}$ in. have all proved suitable. Flat or elongated particles should not be used as they will not be properly embedded in the concrete.

The Institution of Structural Engineers.

A SPECIAL number of "The Structural Engineer", and some of its contents, are liable to be misleading. It is entitled "The Jubilee issue of the Journal of the Institution of Structural Engineers", whereas this journal is in its 37th year only. It is also said to celebrate the jubilee of the Institution of Structural Engineers, whereas it actually commemorates the founding of the Concrete Institute in the year 1908, mainly as a result of the efforts of the late Edwin O. Sachs, at that time Editor of "Concrete and Constructional Engineering" which he had founded two years earlier. Edwin O. Sachs was also the first chairman of the Executive Council of the Institute, and its success was due largely to the energy and ability of that able and versatile protagonist of concrete.

In the year 1912, to the regret (and indeed distress) of some of the founders, the scope of the Institute was widened to include steelwork, and in 1922 its title was changed to the Institution of Structural

Engineers. The Institution has had a successful career in providing a forum for those concerned with the design and erection of structures generally, and the need for a body to represent the concrete industry only has been met by the formation of the Reinforced Concrete Association and by the organisation of lectures and symposia by the Cement and Concrete Association.

The special number gives an outline of the history of the Concrete Institute and the Institution of Structural Engineers and its branches, and articles on research; foundations; docks, dams, and retaining walls; concrete making; reinforced concrete, steel-frame, timber, aluminium, and composite structures; bridges; and clay products and brickwork.

Change of Address.

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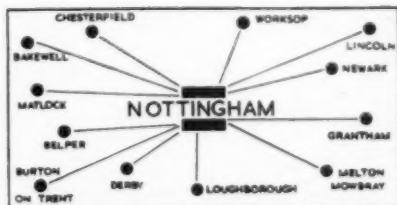
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Concrete at Berkeley Power Station.

MORE than 500,000 tons of concrete are being placed by Messrs. Balfour Beatty & Co., Ltd., and Messrs. John Laing & Son, Ltd. (who are acting as sub-contractors to A.E.I.-John Thompson) at the nuclear power station at Berkeley, Gloucestershire.

At a laboratory tests are made on the cement and aggregates to ensure that they conform to British Standards. Tests for workability are made in a laboratory and also on the site. The permissible limits in the compaction-factor test are 0.86 to 0.94, and cubes are weighed at seven and twenty-eight days to ascertain the density.

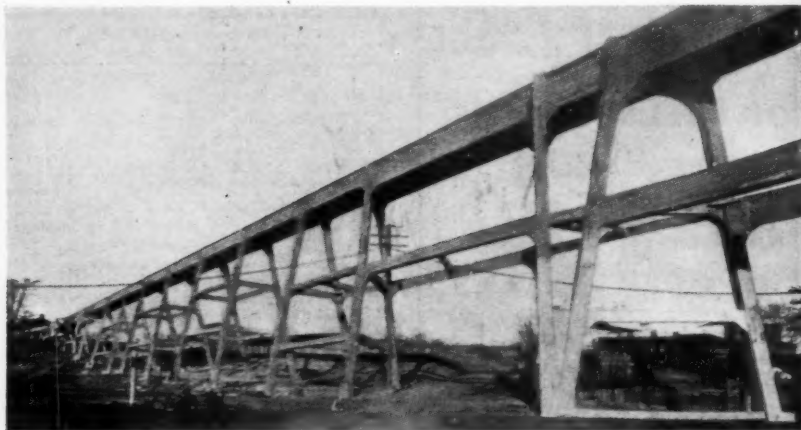
For general purposes a 1 : 7 mixture with a water-cement ratio of 0.62 is used to produce a compressive strength of 3000 lb. per square inch at 28 days. Other mixtures used are 1 : 5.8 with a water-cement ratio of 0.58 and a compressive strength of 4500 lb. per square inch at 28 days and, for the biological shield,

1 : 6½ with a maximum compaction factor of 0.92. Concrete with a compressive strength of 6000 lb. per square inch at 28 days is used for a few purposes. Air-entraining agents are used in some of the concrete.

In order to obtain high density in the biological shields the concrete is placed in lifts of 3 ft. and compacted by pneumatic vibrators (Consolidated Pneumatic type 325) with a frequency of 8000 vibrations per minute and a capacity of compacting 50 cu. ft. of concrete a minute. A density of 150 lb. per cubic foot is obtained with 1 : 6½ mixtures with a water-cement ratio of 0.6. Where a density of 220 lb. per cubic foot is required, as, for example, around the muffs for the control rods, barytes aggregate will be used, and investigations are being made on the use of steel shot as aggregate for the concrete in the cap of the pile; densities up to 350 lb. per cubic foot have been obtained with steel shot.

FIFTY YEARS AGO.

FROM "CONCRETE AND CONSTRUCTIONAL ENGINEERING", SEPTEMBER-OCTOBER, 1908.*



[The illustration is of a viaduct built for the Richmond and Chesapeake Railway, U.S.A. The span of the beams supporting the track varies from 23 ft. to 67 ft. and the greatest height of the viaduct is 70 ft.]

* "Concrete and Constructional Engineering" was published in alternate months until September, 1909.

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